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Journal of the
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# LABORATORY STUDY OF WIND WAVES IN SHALLOW WATER

John C. Hufft,[1] J. M. ASCE
(Proc. Paper 1765)

## SYNOPSIS

A considerable body of information has been accumulated since 1942 on the growth of ocean waves caused by wind action, but very little quantitative information exists on the growth of wind waves in waters where a relatively shallow depth could influence wave growth. The results of a laboratory investigation of the growth of waves in relatively shallow water is presented, showing experimental relationships between wind velocity, fetch, and wave parameters for two different water depths.

## INTRODUCTION

Relationships have been developed by H. U. Sverdrup and W. H. Munk(1) which allow the prediction of waves that will be generated under given conditions of wind velocity, wind duration, and fetch (the length of water surface over which the wind blows). Studies of wind waves by J. W. Johnson at Clear Lake, California,(2) and Abbotts Lagoon, California,(3) and investigations conducted in covered laboratory flumes(4,5) have made possible the extension and slight revision(6) of the original Sverdrup-Munk relationships. Very wide ranges of fetch, wind velocity, and wind duration are covered but the relationships are applicable only under "deep water" conditions, in which the water depth has no effect on the generation of waves.

There exists a need for similar information on the growth of wind waves in waters where the depth could influence their generation. The necessity for this information was pointed out by M. A. Mason in connection with the design of slope protection cover for earth dams.(7) Many similar problems are presented in the design and construction of various waterfront structures for

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1. Lt., Civil Engr. Corps, U. S. Navy, Annapolis, Md.

which information on possible wave action is needed. No method has been developed to allow the prediction of wind waves on bodies of relatively shallow water, nor have the deep water relationships been adapted to wave generation for such intermediate depths.

Many bodies of water exist wherein the depth might influence the formation of waves: Pamlico Sound, North Carolina; Lake Okeechobee, Florida; large parts of the Gulf of Mexico; and numerous reservoirs, lakes and bays.

This paper reports the results of what the author believes to be the first quantitative study conducted under controlled laboratory conditions for wind waves generated in water of intermediate depths.

## Objectives

The principal objectives of this investigation were:

a. To determine whether depth does influence wind wave generation for relatively shallow water depths;
b. To determine which wave parameters, if any, are so influenced and in what manner;
c. To explain any effects of depth, if such were found;
d. And to obtain data which might be of assistance in the future development of relationships between waves generated in relatively shallow water and given weather conditions.

A secondary objective was the measurement of wind set-up. These data are presented for the record, no interpretation or analysis being made. Strong winds pile water on a leeward shore and partially dewater a windward shore. This is known as wind set-up and has frequently been a cause of damage in conjunction with wave action.

## Discussion

### Waves, General Information

Progressive water waves are characterized by an orbital motion of individual water "particles", the combined motions of all such particles resulting in a wave form of successive crests and troughs traveling with a certain velocity. Relationships between various parameters of such waves are given by the following formulae, the derivations of which are readily available:[8]

$$L = C\,T \qquad \text{Eq. 1}$$

$$C^2 = \frac{gL}{2\pi} \tanh \frac{2\pi d}{L} \qquad \text{Eq. 2}$$

in which L is wave length, C is wave velocity, T is wave period, g is acceleration due to gravity, and d is the water depth. Eq. (1) is essentially a definition and is correct for all wave phenomena. Eq. (2) is applicable to all progressive waves in water of constant depth so long as the surface curvature is sufficiently small that surface tension may be neglected. This equation, though derived for waves of small height, is generally applicable to waves of

appreciable height. It is also pertinent to note that the derivation of Eq. (2) assumes a sinusoidal wave shape.

Waves may be considered in three categories: "deep water" waves, waves in water of intermediate depths, and "shallow water" waves. In the case of deep water waves, the ratio of water depth to wave length, by definition, is one-half or greater. Particle motion is approximately circular and the amplitude of the motion decreases exponentially with distance below the surface so that at a depth equal to one-half the wave length, particle motion is practically non-existent (Fig. 1). The bottom, therefore, has no effect upon the waves. With a decrease below one-half in the ratio of depth to wave length, the waves gradually begin to "feel" the bottom. They are no longer deep water waves, but are waves in water of intermediate depth and may finally become, with decreasing depth, shallow water waves. For waves in intermediate depths, the ratio of depth to wave length is between one-half and one-twentieth. Particle motion for such waves is elliptical and extends to the bottom. For shallow water waves, the ratio of depth to wave length is equal to, or less than one-twentieth. Particle motion is in the form of a flat ellipse and extends to the bottom (Fig. 1).

In the cases of deep and shallow water waves, simplifications of Eq. (2) may be used with sufficient accuracy.

Deep water waves ($\frac{d}{L} \geq 0.5$):

$$C_o^2 = \frac{gL_o}{2\pi} \qquad \text{Eq. 3}$$

(Note: subscript "o" refers to deep water waves)

Shallow water waves ($\frac{d}{L} \leq 0.05$):

$$C^2 = gd \qquad \text{Eq. 4}$$

For waves in intermediate depths Eq. (2) must be used in its entirety.

Wind Set-Up

The phenomenon by which water is piled up on a leeward shore is known as wind set-up. In nature it is primarily attributable to the mass transport of water in the direction of wave travel. This is partly due to the drag of the wind on the water surface and partly due to the very nature of waves irrespective of wind. Mass transport due to the latter cause is evident in several wave theories. It is explained by the fact that the velocity of a water particle is not constant but is at a maximum when the particle is at the highest point in its orbit and moving in the direction of the wave. Therefore, each particle

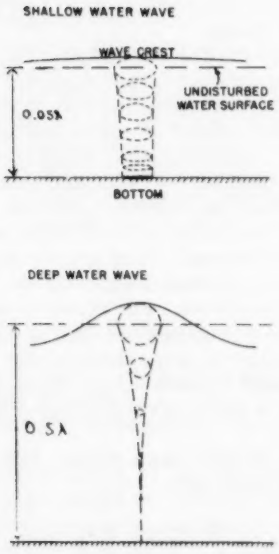



Fig. 1 - Shallow and Deep Water Particle Motions
(After Wiegel and Johnson)

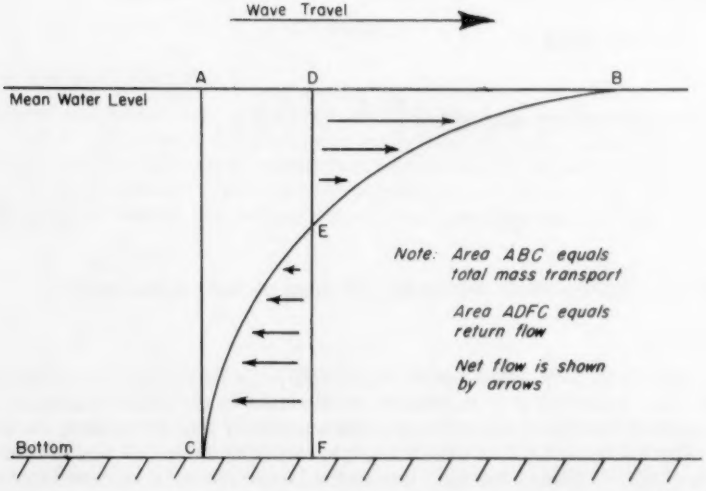



Fig. 2 - Distribution of Mass Transport and Return Flows in a Closed Channel

has a net movement in the direction of wave travel and mass transport results.

When steady state conditions develop in a closed laboratory channel of water responding to a given wind, the mass transport flow must be countered by a return flow in order to maintain equilibrium. The distribution of these flows is shown qualitatively in Fig. 2. A possible result of this return flow will be discussed later.

## Wind Generation of Waves

It is generally accepted that wind energy is transferred to a disturbed water surface by normal and tangential forces. This transfer of energy causes the waves to increase in height, length, and velocity until the wave velocity is approximately equal to that of the wind. Relationships to define this growth in deep water have been developed and are shown on the Fetch Graph (Fig. 3) as modified by Bretschneider.[6] The figure consists of logarithmic plots of non-dimensional parameters from which waves may be forecast using known weather data. Assume that a wind of velocity U acts upon a stretch of water with fetch length F for sufficient time to develop steady state conditions. Then the value of the basic parameter $\frac{gF}{U^2}$ may be computed. It is desired to know the "significant wave height" at the end of the fetch. Therefore, entering Fig. 3 with the computed value of $\frac{gF}{U^2}$, a value for $\frac{gH_o}{U^2}$ is determined from which the significant wave height $H_o$ at the end of the fetch may be calculated. Similarly, other wave parameters may be determined. Wave length may be calculated by the use of Eqs. (1) and (3) in conjunction with the value of $\frac{C_o}{U}$ corresponding to a known value of $\frac{gF}{U^2}$.

Example:

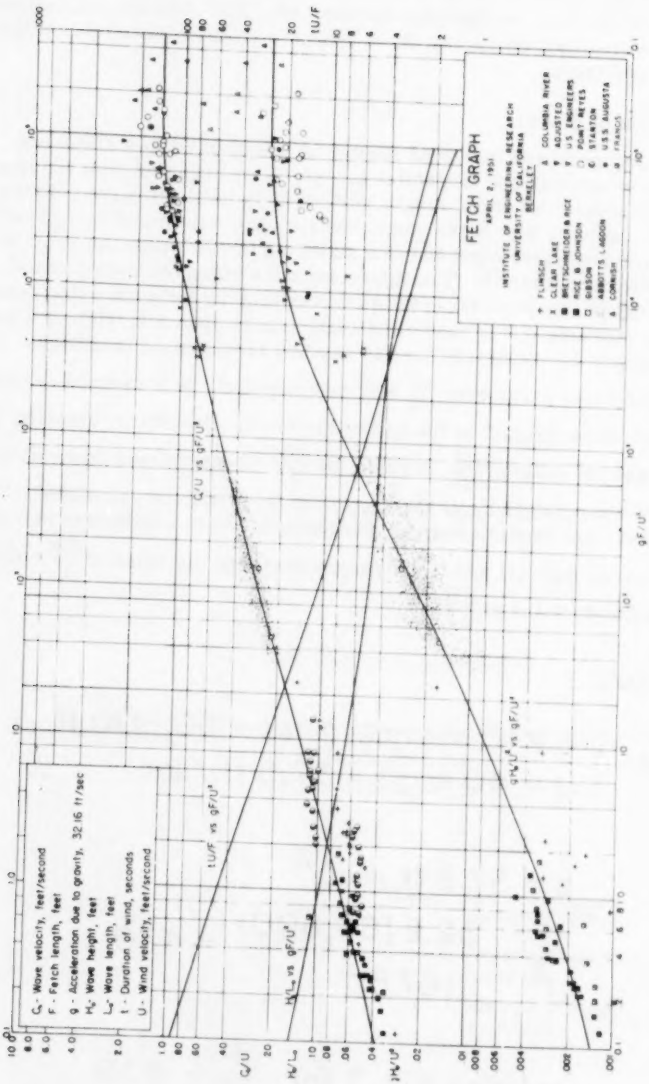
given ( F = 10 nautical miles = 60,800 ft.
( U = 40 knots = 67.5 ft./sec.

$$g = 32.2 \text{ ft./sec.}^2$$

$$\frac{gF}{U^2} = \frac{32.2(60{,}800)}{(67.5)^2} = 430$$

entering Fig. 3 for $\frac{gF}{U^2} = 430$

find $\frac{gH_o}{U^2} = 0.049$


FETCH GRAPH
APRIL 2, 1951
INSTITUTE OF ENGINEERING RESEARCH
UNIVERSITY OF CALIFORNIA
BERKELEY
C - Wave velocity, feet/second
F - Fetch length, feet
g - Acceleration due to gravity, 32.16 ft/sec
H - Wave height, feet
L - Wave length, feet
t - Duration of wind, seconds
U - Wind velocity, feet/second
gF/U²
gt/U
C/U
gH/U²
H/L
gL/U²
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gF/U³¹⁹
gt/U³¹⁸
gH/U³¹⁹
gL/U³¹⁹
gF/U³²⁰
gt/U³¹⁹
gH/U³²⁰
gL/U³²⁰
gF/U³²¹
gt/U³²⁰
gH/U³²¹
gL/U³²¹
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gH/U³²²
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gH/U³²³
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gt/U³⁶¹
gH/U³⁶²
gL/U³⁶²
gF/U³⁶³
gt/U³⁶²
gH/U³

$$\therefore H_o = \frac{0.049(67.5)^2}{32.2} = 6.93 \text{ ft.}$$

Because of the inherent variability of wind waves, the concept of "significant wave height" has been introduced and is defined as the average of the highest one-third of the waves present. In general, it may be stated that the larger waves are the major source of difficulty and therefore this concept appears to have a sound basis. It is now widely accepted and will be used herein.

The results of numerous observations are plotted in Fig. 3 and considerable scatter can be observed. This scatter seems to be inherent in wind wave observation. However, results obtained in actual wave forecasts using this graph in conjunction with meteorological data have been reasonably satisfactory.

The deep water fetch graph will be used as one basis for interpreting the results of this study.

## Experimental Procedure

Water waves were generated by forcing air through a laboratory wave channel two feet wide, three feet deep, and one hundred feet long (Fig. 4ab). The top of the channel was covered with plywood which was made airtight by the use of masking tape. A variable speed exhaust fan was mounted at one end of the channel and an air inlet section with a vertical straightening vane was attached to the other end. The lower plate of the inlet section extended into the channel at the water surface. This served to direct the wind parallel to the water and the end of this plate also established the position of zero fetch. Five electrical conductivity elements extended down through the plywood and into the water at different centerline locations, each element defining a fetch length. The electrical output of each element was dependent upon the depth of submergence in water. The current output, therefore, varied as waves traveled past each element. The varying currents were recorded simultaneously using an oscillograph, thereby providing a record of the waves. Each element was calibrated twice daily to determine the factor relating actual wave height to the movement of each trace on the oscillograph tape. A timing line with half-second interruptions provided a means of measuring wave periods (Fig. 5). Static air pressure taps were located near each end of the channel to determine the pressure drop through the channel, draft gage manometers being used to take these measurements. A pitot tube with a static pressure tap (Prandtl tube) was placed near the middle of the channel at the centerline for wind velocity measurements. Wind velocity profiles for the vertical centerline of the channel were determined for each run. Check measurements taken near each end of the channel showed that the wind velocity was essentially constant longitudinally. No transverse measurements were made, but the wind velocity was believed to be fairly constant across the channel.

At time zero for each run, the exhaust fan and recording oscillograph were started simultaneously. The trace of each element on the oscillograph tape at time zero therefore established the still water level. In order to measure wind set-up of the water surface (and to determine mean water depth at each

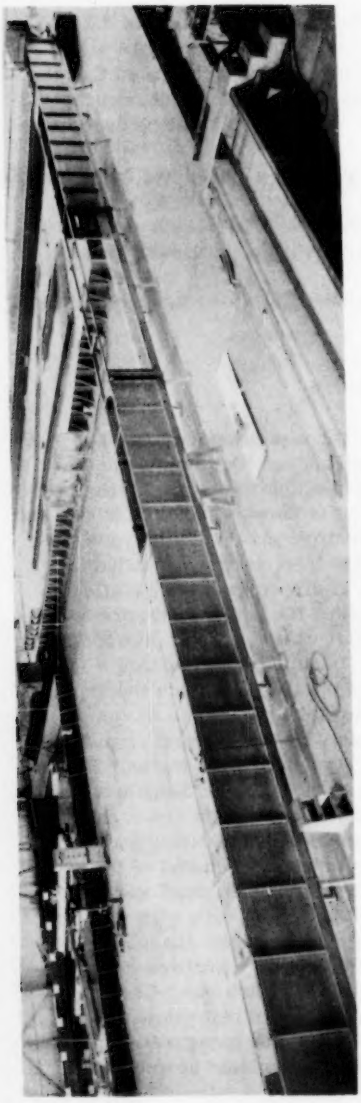

Fig 4a - Wave Channel 100' X 2' X 3'

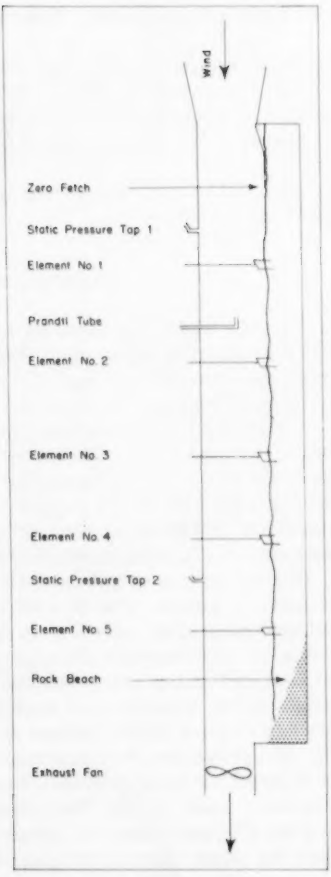


Fig 4b - Schematic Drawing of Channel

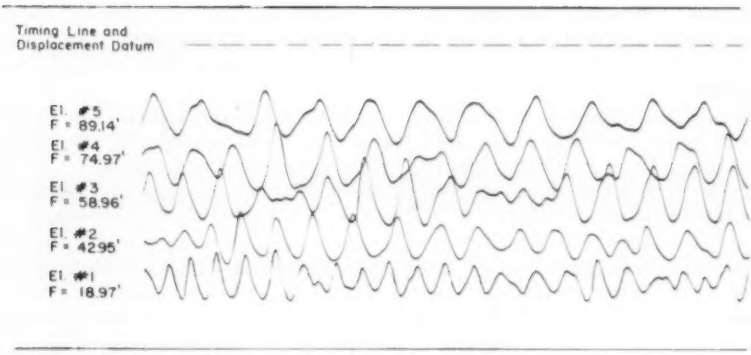


Fig 5 - Portion of a Typical Oscillograph Tape

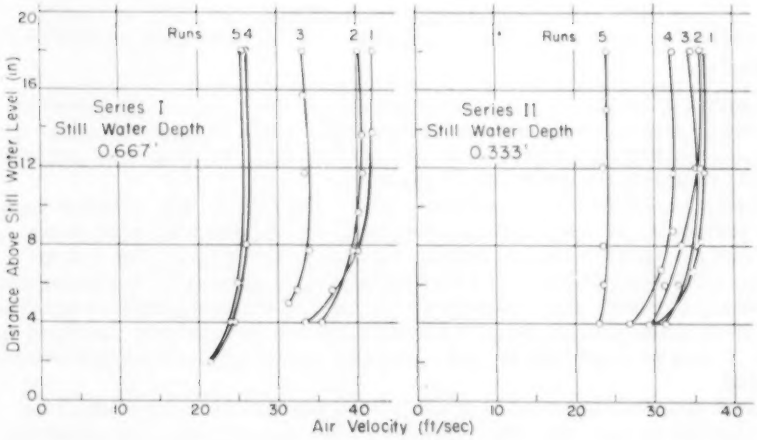


Fig 6 - Air Velocity Profiles

element) the displacement of each trace from its still water position was measured using the timing line as a datum. At steady state the trace for each element was moving according to the waves traveling past the element and the mean displacement had to be measured to the center of the wavy trace. This, naturally, was a possible source of error but results of several readings usually agreed closely.

Two series of tests were conducted. In Series I an eight-inch water depth was used and in Series II a four-inch depth. For these depths, with the wind velocities used, the generated waves were in the deep water and intermediate depth ranges, which was desired. In each series five runs were made using several different wind velocities. For steady state wave conditions, wave heights and periods were measured for each of the five elements defining the five fetches. One hundred waves were analyzed for each element for each run in order to arrive at meaningful values. Significant wave heights were determined by averaging the highest one-third (33) of the hundred waves for each element. A total of five thousand individual wave heights were measured, therefore, to determine significant wave heights. The average wave period for each group of one hundred waves was determined by counting the number of half-second timing pips and dividing the total number of seconds elapsed by one hundred.

Because of the critical effect of depth, it was necessary that the bottom of the channel be level. Precise leveling disclosed that the channel did not deviate from the horizontal by more than five thousandths of a foot. Still water depths were measured to an accuracy of plus or minus three thousandths of a foot.

## Results and Interpretation

### Results

Profiles of centerline wind velocities are shown in Fig. 6. The average velocity of each run was used for computation. Table 1 shows the static air pressure readings at several locations for each run. Readings are given in inches of water below atmospheric pressure.

Wave photographs are presented in Fig. 7 and Fig. 8. Fig. 7 shows wind wave variability in both length and height. It will be noted the wave at the right has partially broken, entraining numerous air bubbles. Fig. 8 is a photograph of the underside of a well-defined wave passing an electrical wave measuring element. For this photograph, the camera was below the waterline (extended) at the glass observation section and the camera axis was tilted upward. It will be noted that the water surface has been extremely disturbed by the wind.

Significant wave heights and average wave periods were determined for each element in each run. Wave lengths were computed from the measured wave periods using Eqs. (1) and (2). Non-dimensional parameters similar to those used by Sverdrup and Munk were computed and the data are presented in Table II.

Plots of water depth-wave length ratio versus fetch length for each run are shown in Fig. 9. It will be noted that waves in the intermediate depth range are generated for all runs.

The non-dimensional data of Table II are presented graphically in Fig. 10 (8 inch water depth) and Fig. 11 (4 inch water depth) on which some of the

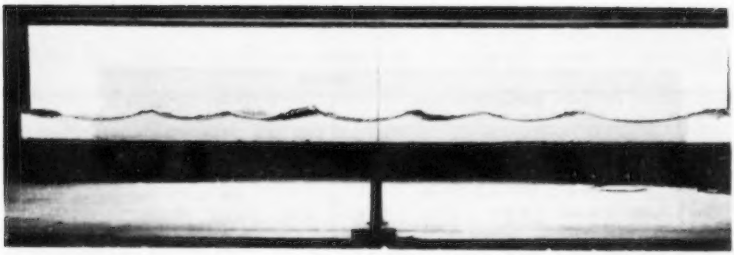

Fig 7 - Wind Waves

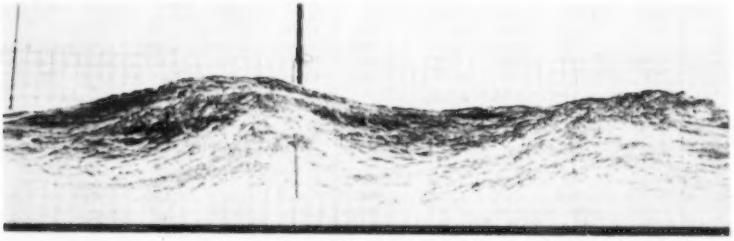

Fig 8 - Underside of a Wind Wave Passing a Wave Measuring Element

Table I

STATIC AIR PRESSURE (GAGE)
(Readings are in inches of water)

| Series and Runs | Tap 1 Distance from Zero Fetch = 16.0' | Prandtl Tube Distance from Zero Fetch = 34.5' | Tap 2 Distance from Zero Fetch = 79.0' |
|---|---|---|---|
| Series I, Run 1 | - 0.42 | - 0.51 | - 0.75 |
| 2 | - 0.42 | - 0.50 | - 0.70 |
| 3 | - 0.30 | - 0.34 | - 0.44 |
| 4 | - 0.18 | - 0.21 | - 0.28 |
| 5 | - 0.18 | - 0.21 | - 0.28 |
| Series II, Run 1 | - 0.34 | - 0.40 | - 0.54 |
| 2 | - 0.34 | - 0.40 | - 0.54 |
| 3 | - 0.34 | - 0.40 | - 0.54 |
| 4 | - 0.28 | - 0.33 | - 0.44 |
| 5 | - 0.17 | - 0.19 | - 0.24 |

Table II

RESULTS

| | Series I, Still Water Depth = 0.667' | | | | | Series II, Still Water Depth = 0.333' | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Sta. #1 Fetch= 18.97' | Sta. #2 Fetch= 42.95' | Sta. #3 Fetch= 58.96' | Sta. #4 Fetch= 74.97' | Sta. #5 Fetch= 89.14' | Sta. #1 Fetch= 18.97' | Sta. #2 Fetch= 42.95' | Sta. #3 Fetch= 58.96' | Sta. #4 Fetch= 74.97' | Sta. #5 Fetch= 89.14' |
| | Run 1, U = 41.3 ft/sec | | | | | Run 1, U = 36.0 ft/sec | | | | |
| d (ft) | 0.637 | 0.650 | 0.679 | 0.699 | 0.728 | 0.312 | 0.339 | 0.353 | 0.369 | 0.377 |
| H (ft) | 0.110 | 0.142 | 0.204 | 0.206 | 0.229 | 0.114 | 0.127 | 0.141 | 0.115 | 0.099 |
| T (sec) | 0.41 | 0.58 | 0.67 | 0.73 | 0.84 | 0.42 | 0.60 | 0.64 | 0.70 | 0.78 |
| L (ft) | 0.86 | 1.70 | 2.22 | 2.54 | 3.14 | 0.87 | 1.57 | 1.75 | 1.99 | 2.26 |
| H/L | 0.128 | 0.084 | 0.092 | 0.081 | 0.073 | 0.130 | 0.081 | 0.080 | 0.058 | 0.044 |
| gH/U$^2$ | 0.0021 | 0.0027 | 0.0038 | 0.0039 | 0.0044 | 0.0028 | 0.0032 | 0.0035 | 0.0029 | 0.0025 |
| gL/U$^2$ | 0.016 | 0.032 | 0.042 | 0.048 | 0.059 | 0.022 | 0.039 | 0.043 | 0.049 | 0.056 |
| gF/U$^2$ | 0.36 | 0.81 | 1.11 | 1.42 | 1.68 | 0.47 | 1.17 | 1.46 | 1.86 | 2.22 |
| | Run 2, U = 40.5 ft/sec | | | | | Run 2, U = 35.2 ft/sec | | | | |
| d (ft) | 0.652 | 0.660 | 0.690 | 0.690 | 0.722 | 0.304 | 0.331 | 0.341 | 0.360 | 0.374 |
| H (ft) | 0.130 | 0.168 | 0.218 | 0.215 | 0.229 | 0.097 | 0.109 | 0.121 | 0.128 | 0.110 |
| T (sec) | 0.46 | 0.61 | 0.70 | 0.78 | 0.86 | 0.40 | 0.54 | 0.61 | 0.72 | 0.75 |
| L (ft) | 1.18 | 1.86 | 2.36 | 2.80 | 3.24 | 0.81 | 1.36 | 1.63 | 2.03 | 2.22 |
| H/L | 0.110 | 0.090 | 0.092 | 0.077 | 0.071 | 0.120 | 0.080 | 0.074 | 0.063 | 0.049 |
| gH/U$^2$ | 0.0025 | 0.0033 | 0.0043 | 0.0042 | 0.0045 | 0.0025 | 0.0028 | 0.0031 | 0.0033 | 0.0029 |
| gL/U$^2$ | 0.023 | 0.036 | 0.046 | 0.055 | 0.064 | 0.021 | 0.036 | 0.043 | 0.053 | 0.058 |
| gF/U$^2$ | 0.37 | 0.84 | 1.16 | 1.47 | 1.75 | 0.49 | 1.12 | 1.53 | 1.95 | 2.32 |
| | Run 3, U = 33.3 ft/sec | | | | | Run 3, U = 35.0 ft/sec | | | | |
| d (ft) | 0.649 | 0.667 | 0.674 | 0.682 | 0.692 | 0.304 | 0.331 | 0.341 | 0.360 | 0.374 |
| H (ft) | 0.125 | 0.150 | 0.186 | 0.182 | 0.171 | 0.106 | 0.117 | 0.122 | 0.127 | 0.111 |
| T (sec) | 0.41 | 0.53 | 0.60 | 0.72 | 0.76 | 0.42 | 0.58 | 0.62 | 0.67 | 0.72 |
| L (ft) | 0.86 | 1.43 | 1.81 | 2.47 | 2.74 | 0.92 | 1.49 | 1.65 | 1.86 | 2.05 |
| H/L | 0.145 | 0.105 | 0.103 | 0.074 | 0.062 | 0.115 | 0.079 | 0.074 | 0.068 | 0.054 |
| gH/U$^2$ | 0.0036 | 0.0043 | 0.0054 | 0.0053 | 0.0050 | 0.0028 | 0.0031 | 0.0032 | 0.0033 | 0.0029 |
| gL/U$^2$ | 0.025 | 0.041 | 0.052 | 0.071 | 0.079 | 0.024 | 0.039 | 0.043 | 0.049 | 0.054 |
| gF/U$^2$ | 0.54 | 1.25 | 1.71 | 2.17 | 2.58 | 0.50 | 1.13 | 1.55 | 1.97 | 2.34 |
| | Run 4, U = 25.3 ft/sec | | | | | Run 4, U = 31.3 ft/sec | | | | |
| d (ft) | 0.655 | 0.663 | 0.667 | 0.672 | 0.681 | 0.316 | 0.335 | 0.344 | 0.354 | 0.364 |
| H (ft) | 0.070 | 0.110 | 0.134 | 0.132 | 0.121 | 0.092 | 0.133 | 0.120 | 0.123 | 0.092 |
| T (sec) | 0.34 | 0.44 | 0.52 | 0.60 | 0.64 | 0.38 | 0.54 | 0.60 | 0.62 | 0.64 |
| L (ft) | 0.60 | 1.01 | 1.40 | 1.84 | 2.06 | 0.76 | 1.36 | 1.59 | 1.65 | 1.75 |
| H/L | 0.116 | 0.109 | 0.096 | 0.072 | 0.059 | 0.121 | 0.098 | 0.076 | 0.075 | 0.053 |
| gH/U$^2$ | 0.0035 | 0.0055 | 0.0067 | 0.0066 | 0.0061 | 0.0030 | 0.0044 | 0.0039 | 0.0040 | 0.0030 |
| gL/U$^2$ | 0.030 | 0.051 | 0.071 | 0.093 | 0.104 | 0.025 | 0.045 | 0.052 | 0.054 | 0.058 |
| gF/U$^2$ | 0.96 | 2.16 | 2.97 | 3.78 | 4.49 | 0.62 | 1.41 | 1.94 | 2.47 | 2.92 |
| | Run 5, U = 25.3 ft/sec | | | | | Run 5, U = 23.5 ft/sec | | | | |
| d (ft) | 0.658 | 0.660 | 0.673 | 0.678 | 0.676 | 0.322 | 0.336 | 0.341 | 0.347 | 0.355 |
| H (ft) | 0.065 | 0.091 | 0.132 | 0.129 | 0.123 | 0.062 | 0.074 | 0.091 | 0.093 | 0.085 |
| T (sec) | 0.32 | 0.45 | 0.50 | 0.58 | 0.63 | 0.32 | 0.45 | 0.52 | 0.55 | 0.60 |
| L (ft) | 0.54 | 1.04 | 1.30 | 1.70 | 1.98 | 0.52 | 1.00 | 1.29 | 1.40 | 1.57 |
| H/L | 0.120 | 0.088 | 0.101 | 0.076 | 0.062 | 0.119 | 0.074 | 0.070 | 0.066 | 0.054 |
| gH/U$^2$ | 0.0033 | 0.0046 | 0.0066 | 0.0065 | 0.0062 | 0.0036 | 0.0043 | 0.0053 | 0.0054 | 0.0049 |
| gL/U$^2$ | 0.027 | 0.052 | 0.066 | 0.086 | 0.101 | 0.030 | 0.058 | 0.075 | 0.082 | 0.092 |
| gF/U$^2$ | 0.96 | 2.16 | 2.97 | 3.78 | 4.49 | 1.11 | 2.51 | 3.44 | 4.37 | 5.20 |

d = mean water level
H = significant wave height
T = average wave period
L = wave length based on T
H/L = wave steepness
U = average wind velocity
F = fetch length
g = acceleration due to gravity

curves of the deep water Fetch Graph (Fig. 3) are also shown. Figs. 10 and 11 include only the portion of Fig. 3 for the range $\frac{gF}{U^2}$ between 0.1 and 10, all of the data of the investigation being in this range. Also, a new deep water plot $\frac{gL_o}{U^2}$ vs. $\frac{gF}{U^2}$, has been added in order to compare lengths of waves generated in deep water with those generated in intermediate depths. This plot is determined by manipulating Eq. (3) to convert the known values of $\frac{C_o}{U}$ (Fig. 3) to the desired $\frac{gL_o}{U^2}$ for each value of $\frac{gF}{U^2}$, viz:

$$C_o^2 = \frac{gL_o}{2\pi} \qquad \text{Eq. 3}$$

or

$$L_o = \frac{2\pi C_o^2}{g}$$

substituting for $L_o$ in $\frac{gL_o}{U^2}$:

$$\frac{gL_o}{U^2} = \frac{g}{U^2}\left(\frac{2\pi C_o^2}{g}\right) = 2\pi\left(\frac{C_o}{U}\right)^2$$

The data on wind set-up are not presented separately as such. The water depths given for each element in each run in Table II may be correlated with wind velocity to allow determination of various shear stress coefficients. Table I showing static air pressure readings should assist in such an endeavor.

Interpretation of Results

Examination of Figs. 10 and 11 discloses that nearly all of the wave steepness (H/L) curves for intermediate depths follow a well-defined pattern, which may be divided into three important zones:

a. For the first (left) zone of each curve, the rate of decrease of wave steepness with increasing fetch $\frac{(gF)}{U^2}$ is slightly greater than the rate of decrease in steepness occurring in deep water;
b. For the second zone, the rate of decrease of wave steepness becomes less and in some cases wave steepness increases with increasing fetch;
c. For the last zone of each curve, wave steepness decreases rapidly with increasing fetch.

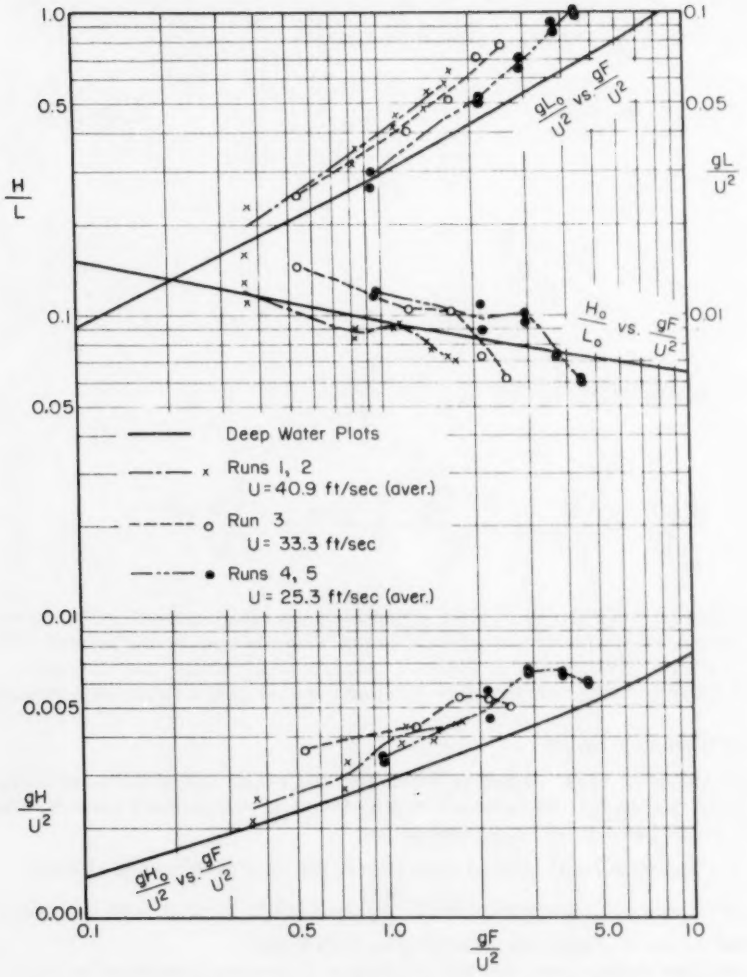


Fig. 10 - Comparison of intermediate Depth Wind Waves with Deep Water Wind Waves
Series I, Still Water Depth Equals 0.667 Feet

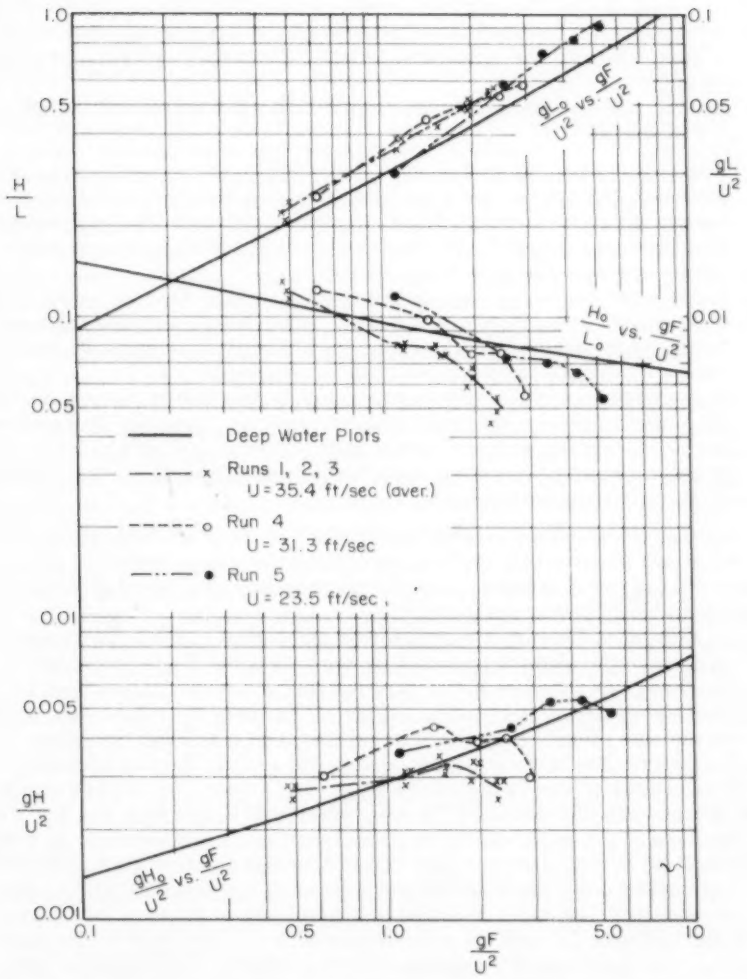


Fig. 11 - Comparison of intermediate Depth Wind Waves with Deep Water Wind Waves Series II, Still Water Depth Equals 0.333 Feet

Inasmuch as wave steepness is dependent upon wave height and wavelength, it is desirable to see which of these two quantities varies as does the steepness curve. In each case, it will be seen that the curves of wave length for waves generated in intermediate depth are approximately the same as those for deep water waves, though the former are slightly higher. With respect to height, however, it will be seen that the $\frac{gH}{U2}$ curves may be divided into three zones similar to those of the steepness curve.

In all cases, the second and third zones of both the wave height and steepness curves occur in the intermediate depth range (see Fig. 9). Therefore, it must be concluded that the influence of depth caused the curves to differ from the deep water curves.

A physical interpretation of these zones in the wave height and steepness curves is desirable.

a. In the first zone, the waves had a water depth-wave length ratio near one-half. Therefore, the growth of the waves should have followed the curves for waves generated in deep water; such was the case, except that the intermediate depth steepness curves decrease at a slightly faster rate than the deep water curves.
b. In the second zone, the influence of depth began to have an appreciable effect on the generation of the waves. Wave height began to increase more rapidly than would have been the case in deep water. The waves tended to steepen and probably became unstable.
c. In the third zone, the waves began to lose height due to the instability developed in the second zone. Partial breaking occurred (this was observed during these experiments) and the waves lost energy. It is likely that this partial breaking would have continued until the waves were stable under the existing conditions.

It was therefore established that differences between wave generation in deep water and in relatively shallow depths do exist. It is desirable to determine what physical factors cause the differences. Several possibilities are suggested in the following paragraphs.

It has been pointed out that a return flow must exist in a closed channel of water to counter the mass transport flow due to waves. Since the mass transport flow is an inherent factor in progressive waves, it should not be considered as modifying the waves. On the other hand, the return flow could have a noticeable effect. It is a well known fact that a current traveling against a progressive wave train can modify the waves. Such an opposing current can cause an increase in wave height and a decrease in wave length by the phenomenon of refraction. In deep water, the return flow has a low velocity because of the relatively large cross-sectional area for flow. In water of intermediate depths, however, the return flow has a relatively higher velocity and consequently there is more likelihood of a refractive effect. In these experiments, the rapid increase in height (second zone) is more noticeable in Series I with an eight-inch depth than with Series II with a four-inch depth. It would seem that the opposite should be true if the refractive effect of return current had caused the rapid increase in height because the velocity of return flow should have been greater in the four-inch depth of water. Also, no decrease in wave length due to refraction was evident. However, these two facts cannot be considered conclusive and the refractive effects of return flow must be considered as a possible cause of the differences between wave

generation in deep water and wave generation in relatively shallow water.

A second possible cause of these differences is the change in wave characteristics as the waves begin to "feel" the bottom. This change can best be described by referring to waves of a given length moving into shoaling water unassisted by wind. Such a condition is analogous to wind waves being generated in water of constant, intermediate depths in that both represent situations wherein the ratio of depth to wave length decreases with distance traveled by the waves. In the case of the "unassisted" waves, the water depth decreases rapidly, the wave length remaining almost constant, whereas in the case of the wind waves, the length increases rapidly while the depth remains practically constant. It may logically be expected that the net result on wave shape would be the same for both situations. Moving into shoaling water unassisted by wind, waves decrease slightly in steepness as they first begin to "feel" the bottom and then, as the ratio of depth to wave length becomes approximately one-quarter, the steepness begins to increase rapidly.(9) A similar change in steepness is noted when examining the experimental curves of Figs. 10 and 11. That is, the rate of decrease of steepness is greater in the first zone of each experimental curve, than for the deep water curves. In this zone, the waves had just begun to "feel" bottom. As the ratio of depth to length continues to decrease, wave steepness begins to decrease at a slower rate (second zone) and in some cases, steepness actually increases. Therefore, it seems likely that change in wave shape due to a decrease in the ratio of water depth to wave length explains, at least partially, the differences in generation of waves in intermediate depths as compared to generation in deep water.

It has been explained that the loss in wave height with increasing fetch (third zone) is due to a partial breaking of the unstable waves. It remains to be seen why the waves became unstable and consequently broke. The breaking of waves is a very complex question, particularly when a wind is blowing. Therefore, it must suffice for the present to say that the waves broke probably because of a combination of reasons including the following:

a. The wind partially blew the tops from the waves.
b. The wave height was appreciable relative to the water depth.
c. The waves reached a critical steepness ratio. It should be pointed out that this critical value may not be the same as that for waves in deep water unaffected by wind. In the latter case, critical steepness is reached when the crest angle is $120^0$, but the possibility exists that waves in intermediate depths, subject to wind action, will break before this crest angle is reached.

## CONCLUSIONS AND RECOMMENDATIONS

### Conclusions

The principal objectives of this investigation were:

a. To determine whether depth does influence wind wave generation for relatively shallow water depths;
b. To determine which wave parameters, if any, are so influenced and in what manner;
c. To explain any effects of depth, if such were found;
d. And to obtain data which might be of assistance in the future development of relationships between waves generated in relatively shallow water and given weather conditions.

The results of this investigation allow the following conclusions:

a. Water depth does have a decided influence upon wind wave generation in relatively shallow water depths.
b. Wave steepness and wave height are affected by depth when the waves are in the intermediate depth range. Experimental wave lengths do not appear to differ greatly from those which would be generated under deep water conditions. This report does not include comparisons of either wave period or wave velocity for intermediate depth and deep water conditions. Such comparisons were made, however, using the information presented in Table II, and showed that wave periods and wave velocities, for the conditions investigated, were approximately the same as for deep water conditions.
c. The differences between waves generated in intermediate depths and those generated in deep water are probably caused by the following factors (explained fully in section "Interpretation of Results"):

   (1) Refractive effects of return flow caused by mass transport flow (this may not be important in irregularly-shaped bodies of water),
   (2) Change in wave shape as the waves begin to "feel" the bottom,
   (3) Partial breaking of waves caused by the wind actually blowing the tops from the waves,
   (4) Partial breaking of waves occurring when wave height becomes appreciable compared to depth,
   (5) Partial breaking of waves caused by waves reaching critical steepness.

d. The data presented should be of assistance to future investigators in the development of relationships for wind waves generated in water of intermediate depth.

The secondary objective of this investigation was the measurement of wind set-up. These data are available in Table II.

Recommendations

Continued investigation of the wind generation of waves in water of intermediate depth is recommended. It is particularly desirable that prototype investigations be undertaken, as well as further laboratory experiments, in order to be able to include conditions representing wide ranges of water depth, fetch length, and wind velocity. Wind duration does not seem to be of primary importance for intermediate depths because steady state conditions are reached in a relatively short period of time compared to usual storm durations.

## ACKNOWLEDGEMENT

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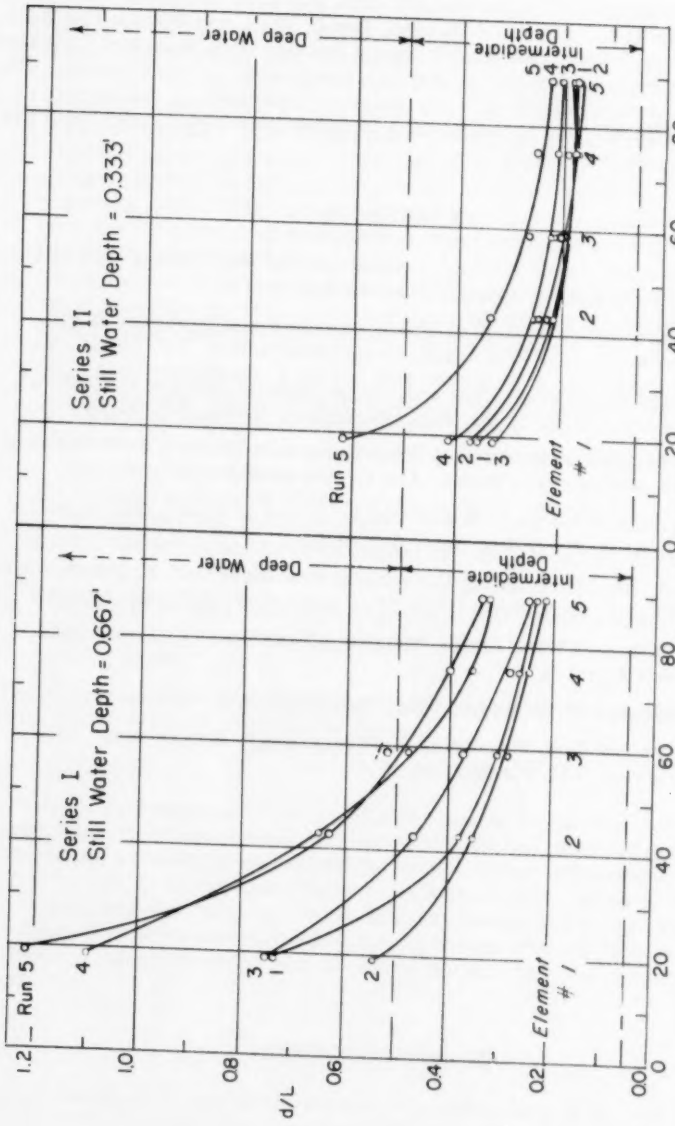


Fig. 9 - Variation of Water Depth - Wave Length Ratios with Fetch

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WATERWAYS AND HARBORS DIVISION
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# IRRAWADDY RIVER SYSTEM OF BURMA[a]

Henry R. Norman,[1] M. ASCE
(Proc. Paper 1766)

## ABSTRACT

This paper is a presentation of factual data regarding transportation on the Irrawaddy River System of Burma up to 1953. It includes the basic conditions of the system pre-war, the war-time disruption, and the post-war recovery measures. It should be of interest to waterway engineers to learn of commercial use of the "Road to Mandalay."

## INTRODUCTION

Burma, although not more than about 262,000 square miles in area, or approximately the size of Texas, is one of the most important countries of Southeast Asia. Blessed by nature with the heavy monsoon rains and the flat fertile river valleys and large deltas of heavy soils, it has long been considered one of the major rice graneries of the world. Teak abounds in the jungle forests on the hills and mountainsides; oil and such important minerals as gold, silver, tin, lead, wolfram, antimony, and precious sapphires, rubies, and jade are contained in her good earth; and rubber plantations dot the countryside in the Irrawaddy Delta and in the peninsular section bordering Thailand (Siam). Because of the abundance of these products, Burma has long been an important supplier in the world market, and the transport of these commodities to and from its seaports has been a major factor in the economy of the country.

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1766 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 4, September, 1958.

a. This paper was prepared for presentation at the 1953 meeting of the ASCE and was subsequently lost until this year. The author particularly invites discussion that will bring the subject matter up-to-date.

1. Cons. Engr., Brown & Root, Inc., Houston, Tex.

## General Geography

Burma lies in a natural basin. As shown in accompanying Figure 1, it is rimmed on the west, north, and east by the mountain ranges separating it from East Pakistan, Assam, Tibet, China, Indo-China, and Thailand, and is bounded on the south by the Andaman Sea and the Bay of Bengal. The rivers in general flow from north to south, from the Himalaya Mountains to the sea, separated one from another by high rugged hills and mountains. Referring to Fig. 2, it is to be noted that the major rivers from west to east are: the Kaladan River with the seaport of Akyab at its mouth; the Irrawaddy River with the seaports of Bassein and Rangoon in its delta; the Sittang River which empties into the Andaman Sea near Rangoon; and the mightly Salween River which is one of the largest systems in the world, with the seaport of Moulmein near its mouth. Over 5,000 miles of commercially navigable waterways are provided by these rivers, and there are several thousand additional miles used by country craft. Nearly the mileage of the Mississippi River system is available in this small area. The lower reaches and delta areas of these streams constitute the major agricultural areas of the country, and in these deltas the labyrinth of streams and creeks makes rail and highway transport impracticable. Water is the only cheap and convenient means of transport for the large volume of rice, consumer goods, and passenger travel within the 10,000-square-mile Irrawaddy Delta, and also within the Kaladan Delta of more than 1,000 square miles.

## Historical Development

As is common in the historic development of all countries, the rivers and waterways have been of prime importance, — first, because of the fertility of the river valleys, and second, due to the cheap means of transport by water. Burma's history follows this same pattern. Since population is distributed in the river valleys - essentially self sufficient in production — and because of the natural obstructions to rail and highway imposed by the jungles, there had been little need for travel between watersheds, and trade generally followed the river patterns. Rail and highway construction has lagged and the major routes provided by these facilities have been principally to connect the two largest cities, — Mandalay, the former city of Kings with its multitude of shrines and pagodas, and Rangoon, the Capital and major seaport.

## Description of Waterways System

The waterway most important to the economic development of Burma has been the Irrawaddy River System, — the "Road to Mandalay" immortalized by Kipling. This river with its principal tributary, the Chindwin River, drains over 55 per cent (143,900 square miles) of Burma, and these two streams are the principal avenues of riverine commerce. As shown on Fig. 3, commercial transportation on the Irrawaddy River proper can be maintained for about 980 miles, about 855 miles from Rangoon to Bhamo throughout the year, and 125 miles from Bhamo to Myitkyina for seven months each year. Henzada, about 130 miles above Rangoon, is considered as the head of the delta and the beginning of the river proper; it is also the upstream limit of the tidal effect. In the delta section below Henzada there are about 2,000 miles of

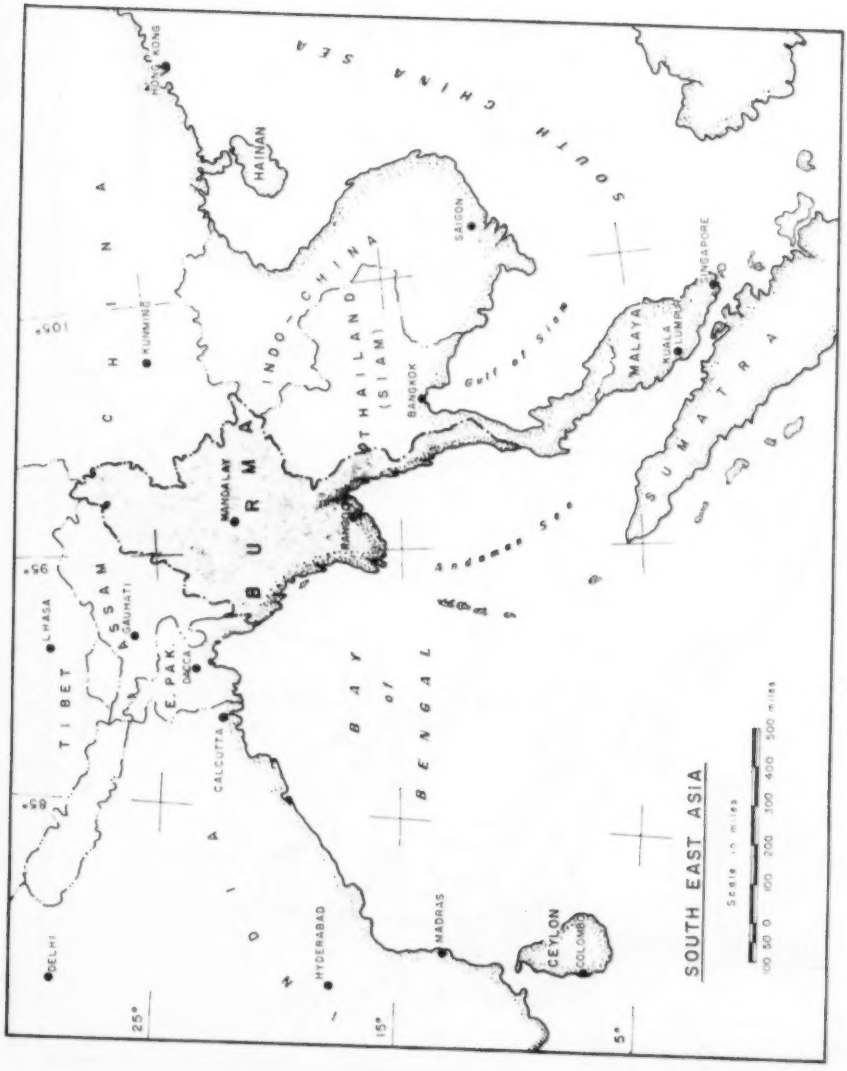


Fig. 1. Southeast Asia showing general location of Burma.

**Fig. 2. Upper Burma, showing principal rivers and river ports.**

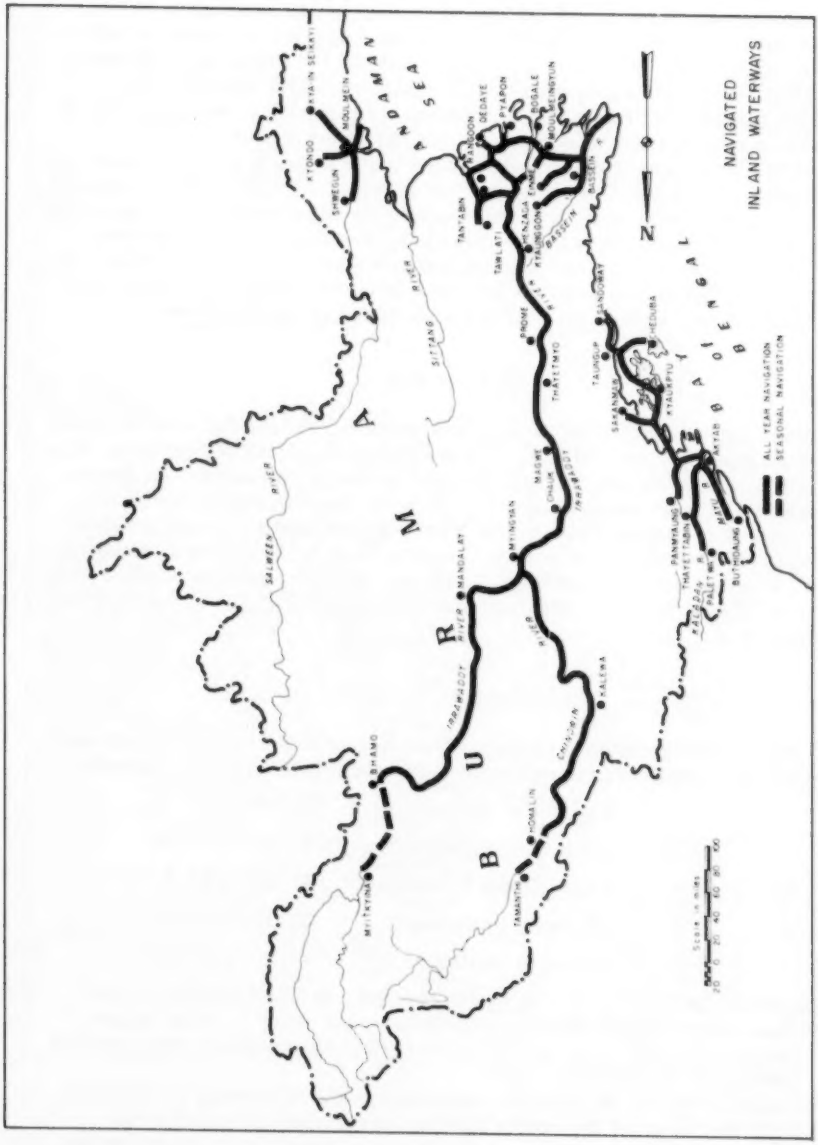


Fig. 3. Commercially navigated inland rivers and waterways of Burma.

commercially navigable waterways and thousands of other miles of streams which are used by the sampans and country boats. This intricate maze of natural and commercial waterways are as shown on Fig. 4. The Chindwin River is open for commercial navigation throughout the year from its confluence with the Irrawaddy near Pakokku to Homalin, for a distance of about 400 miles; however, launches can reach Tamanthi (57 miles above Homalin) during the monsoon season, and small launches go as far upstream as Singaling Hkamti (123 miles above Homalin) throughout the year (see Fig. 2). Homalin is considered as the head of commercial navigation.

Both the Irrawaddy and Chindwin Rivers above Henzada in the commercially navigated reaches are typically alluvial streams. They are highly braided and heavily beset with sand bars which are in constant movement. Especially in the section from Henzada to Mandalay it is necessary to be on constant guard for channel movements, shifting and new sand bars, and for snags. In the delta area below Henzada, the labyrinth of cross channels and distributaries are sinuous waterways with complex tidal and river currents.

## Hydrography

Rainfall in Burma follows the monsoon pattern. All rainfall occurs in the period between May and October, with the concentrated fall during June, July, and August. From October to May there is normally no rainfall. In Burma the maximum annual rainfall varies up to more than 225 inches per year; however, in the drainage basin of the Irrawaddy the annual rainfall ranges from about 100 inches at Rangoon in the delta area, to 33 inches at Mandalay, and to 160 inches in the headwaters. Average annual variations in river stage at Prome (235 miles above Rangoon) and Mandalay are 37.3 feet and 31.7 feet, respectively.

## Limitations to Navigation

On the Irrawaddy River the lowest levels are usually reached in February and March and the drafts of vessels are at present limited to the following:

| | |
|---|---|
| Rangoon to Prome | 5'-0" |
| Prome to Yenangyaung | 4'-9" to 5'-0" |
| Yenangyaung to Mandalay | 4'-3" to 4'-9" |
| Mandalay to Bhamo | 3'-6" to 4'-3" |
| Bhamo to Myitkyina | 2'-6" to 3'-0" |

In general, from December to March the river is 0 to 5 feet above the lowest level and from mid-June to mid-October the river is 20 to 30 feet above the lowest level. It is usually necessary to have separate high and low water landings at the riverine ports.

The usable width of the river is controlled by the sharpness of the bends which limit the length of the vessel and the overall dimensions of a tow which can ascend or descend the river. Between Rangoon and Mandalay the permissible overall dimensions for a tow of 4 to 4.5 feet of draft during the low water season are:

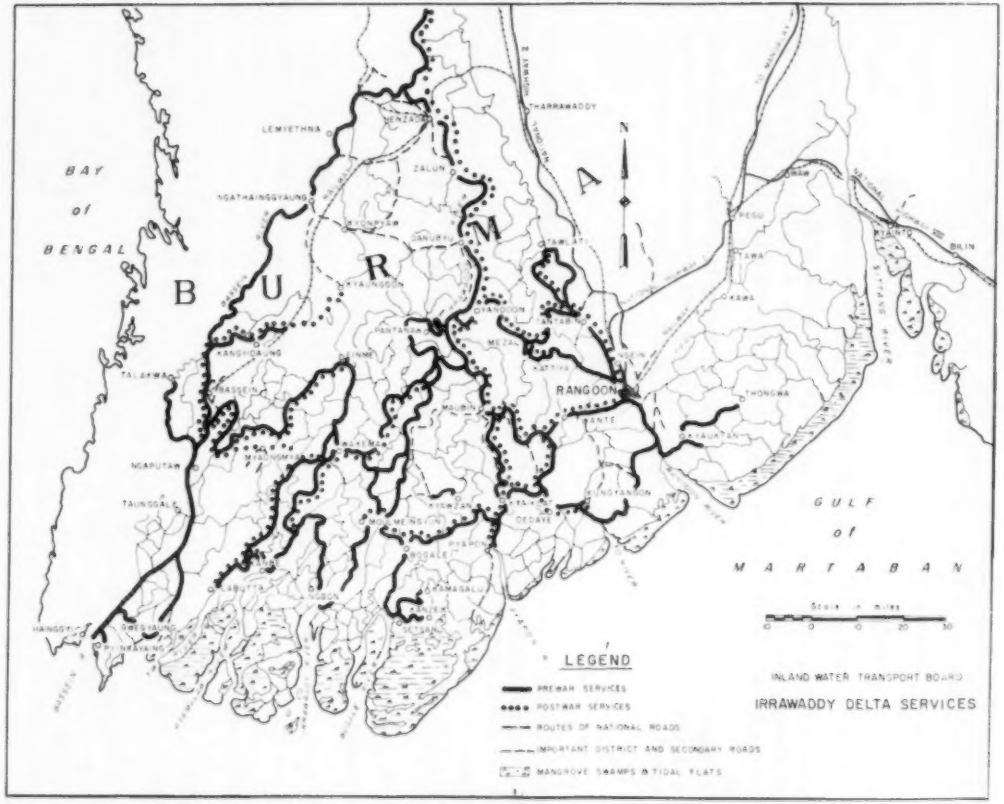


Fig. 4. Natural and commercial waterways of the Irrawaddy Delta.

Width - Good Conditions - 200 feet

Normal Conditions - 150 feet

At bad shoal crossings - 100 feet with difficulty

Length - Up to 450 feet in normal or fairly adverse conditions.

Between Mandalay and Bhamo the overall dimensions of a tow of 3.5 to 4.0 feet of draft during low water period are approximately:

Width - Normal Conditions - 110 feet

At bad shoal crossings - 70 feet with difficulty

Length - Up to 400 feet in normal or adverse conditions.

Between Bhamo and Myitkyina the length and width of the operating craft are limited by the sharp and narrow bends between rocks in the first defile. The probable maximum for this reach of the Irrawaddy River is 150 feet by 50 feet.

The maximum draft up the Chindwin River during low water periods is 2 feet 6 inches to 3 feet. The river is about 500 yards wide at Homalin during low water.

## River Ports

The principal river ports on the Irrawaddy River and their respective distances from Rangoon are as follows:

| | | | | | |
|---|---|---|---|---|---|
| Yandoon | 76 river miles | | Chauk | 410 river miles | |
| Henzada | 131 " | " | Myingyan | 472 " | " |
| Prome | 235 " | " | Mandalay | 552 " | " |
| Thayetmyo | 276 " | " | Katha | 725 " | " |
| Magwe | 346 " | " | Bhamo | 802 " | " |
| Minbu | 349 " | " | Myitkyina | 927 " | " |
| Yenangyaung | 357 " | " | | | |

Only a few of these localities have handing facilities which usually consist of one or two flats or barges. In most cases vessels moor alongside the river bank. There are generally two separate landing stages, one used during the high water period and another during the dry season. They are often separated by a considerable distance, and in many cases no proper road exists between the landing stage and the town. Cargo storage facilities are generally not available, and the cargo is stored either on the landing flats and barges, some of which are roofed, or openly on the river banks. The more important of these river ports are:

Henzada: At the head of the Irrawadda Delta, it is the junction of the railway lines to Kyangin and Bassein. The Prome to Rangoon railway line has a connection to Tharrawaw and there is a ferry crossing between Tharrawaw and Henzada.

Prome: The rail head of the Rangoon to Prome branch of the Burma Railways.

Chauk: The center of the oil fields. There are facilities here for handling petroleum shipments in bulk and in barrels.

Mandalay: The principal city of upper Burma and the rail and highway center of northern Burma.

The principal localities served by commercial vessels on the Chindwin River and their respective distances from the confluence with the Irrawaddy River are:

| | | | |
|---|---|---|---|
| Monywa | 54 river miles | Paungbyin | 317 river miles |
| Kalewa | 195 " " | Homalin | 381 " " |
| Mawlaik | 239 " " | Tamanthi | 438 " " |
| Sittaung | 295 " " | | |

As there is no railway in the Chindwin Valley north of Monywa and a very limited length of highways, riverine craft provide the principal means of transportation for the hinterland of the Chindwin Valley.

## Kalewa Coal Deposits

Near Kalewa explorations are in progress to determine the extent of a coal seam, the outcrop of which has been observed at several locations over a distance of sixty miles and which appears to have a fairly uniform thickness of about ten feet. The development of this coal deposit will increase the traffic on the river materially. At the present time Burma imports from India all coal used in the country.

## Types of Craft and Commerce

Transport on the Irrawaddy system can be divided into three classes: craft operated by the Government, craft operated by others as private and contract carriers, and the multitude of private sampans and country boats. Since the country craft are not licensed nor are their cargoes reported, it is not possible to determine the magnitude of the riverine commerce in which they are engaged; however, judging from the internal and foreign trade commerce of the seaports, it is apparent that these craft carry a large proportion of the trade. It has been estimated that the commerce of the sampans and country craft may exceed that of the Government and privately owned vessels by as much as three and four times. These country craft are small with the majority in the 1- to 10-ton class, but some of them engaged in the transport of rice paddy to the mills may range up to as much as 50 tons. In addition, large tonnages of teak and bamboo are rafted on the rivers and delta streams, carried along by the river and tidal currents.

The Inland Water Transport Board, a Government organization constituted in 1946, operates the largest fleet of riverine craft on the Irrawaddy and Chindwin Rivers. The Irrawaddy Section of the Inland Water Transport Board took over the organization of the former Irrawaddy Flotilla Company when the latter was nationalized. Besides the services on the Irrawaddy and Chindwin

Rivers the Inland Water Transport Board operates a fleet in the Moulmein District, a lighterage fleet of about 10,000 tons capacity in the Port of Rangoon, where it handles approximately 30 to 40 per cent of the lighterage there, and also in the Arakan Section. The government-owned cargo fleet in the Arakan area of western Burma is operated through its agent, the Arakan Flotilla Company, a subsidiary of the British India Steam Navigation Company.

The average annual prewar commerce of the Irrawaddy Flotilla Company is estimated at 1,200,000 tons of cargo and 8,200,000 passengers. During the last two fiscal years (October 1 to September 30 periods) the commerce carried by the Inland Water Transport Board was as follows:

| | 1950-1951 | 1951-1952 |
|---|---|---|
| Cargo, long tons | 641,000 | 662,000 |
| Passengers | 2,530,000 | 3,080,000 |

Insurgency materially reduced riverine commerce during the fiscal years 1948-1949 and 1949-1950, and at the present time prevents the operations from proceeding normally. Vessels can operate only during the daylight hours and must remain overnight at a garrisoned river port. The pilot houses and engine rooms of the craft are protected by armor plate and they proceed under military guard and protection, often in convoys under additional naval protection. Insurgency also interferes with conservancy operations by the destruction of markers and channel buoys, thus the vessels often are required to feel their way along the shifting river channels.

Prior to the war, the fleet of the Irrawaddy Flotilla Company consisted of a total of 636 vessels of all descriptions, including 246 powered craft of all types and 372 various barges. This fleet essentially was completely destroyed during the war. By 1950, the fleet of the Inland Water Transport Board numbered about 556 vessels; however, of this number only 19 powered craft and 86 barges were salvaged and reconditioned prewar vessels. The balance consisted of 84 powered vessels and 50 barges of new construction together with some 300 craft of the Military Directorate of Transport. The total gross tonnage of the powered craft amounted to only 22,000 tons as against approximately 49,000 tons of the prewar fleet. The corresponding figures for barges are 40,000 tons and 191,000 tons. Accompanying Figs. 5 to 10 inclusive show typical craft used in the river and delta services of this time.

The development of the original type of vessels used on the rivers of Burma was controlled by the critical depths and widths of the streams, intensity and complexity of the currents and other hydraulic factors, in addition to the particular requirements of commerce. On the Irrawaddy River in the Rangoon to Mandalay trade, the vessel was a large cargo-passenger steamer type, usually equipped with stern or side paddle wheels. These vessels varied in size up to 326 feet in length and 46 feet (plus 30 feet for side paddle boxes) in width. The vessels operated alone or with two barges in tow, one at each side of the ship. Barges were usually 250 feet by 34 feet. From Mandalay to Bhamo, smaller steamers of the same type were used, about 235 feet in length and 30 feet (plus 20 feet for side paddle boxes) in width. From Bhamo to Myitkyina and on the Chindwin River to Homalin similar vessels were used; upstream from Bhamo the river limits the vessel size to about 150 feet by 50 feet.

Fig. 5. Three-Quarter Deck Paddle Steamer Towing Cargo Flats.

Fig. 6. Three-Quarter Class "M" Paddle Steamer Towing Two Oil Flats.

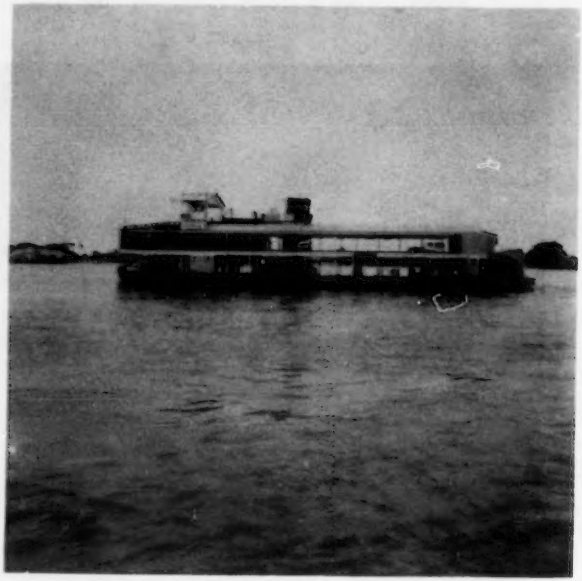

Fig. 7. Quarter Wheeler Built For Chindwin River Service now used on Prome to Mandalay Run.

Fig. 8. "S" Class Single Screw Launch used in Delta Services.

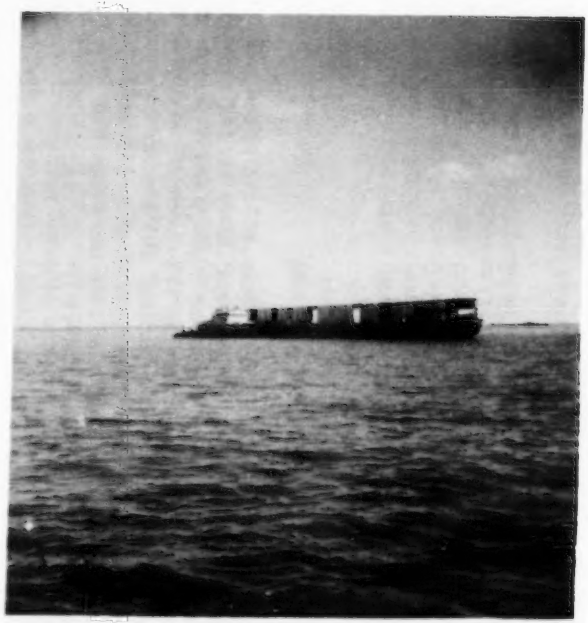

Fig. 9. "A" Type Tug Towing Cargo Flat.

Fig. 10. Prewar Irrawaddy Flotilla Co. Cargo Barge.

In the labyrinth of small streams of the delta sections the complexity of currents resulting from tides ranging up to about 19 feet and the extreme sinuosity of the channels prescribed craft of different type. These were small fast screw-propelled double decked vessels which operated alone without tow.

The prewar Irrawaddy Flotilla Company introduced diesel powered craft as early as 1930. A transition from steam to diesel power is being carried out in the present craft which are in good condition, and diesel power is being used mostly for the new fleet. Most of the steam-powered paddlers were converted to oil firing.

## Fleet Improvement Program

In order to replace the military craft which were too expensive in commercial operation, and to rebuild the fleet to approximately its prewar strength, the Inland Water Transport Board adopted a 3-year program of acquisition of new vessels estimated to cost about $4,200,000. This program is the initial stage of a 10-year program of complete fleet rehabilitation estimated to cost about $31,600,000.

The Inland Water Transport Board is considering whether to continue its ship-building program along the lines of the prewar fleet or whether to introduce a pusher type towboat similar to those used in the United States and South America. The degree of efficiency of a push towboat on the Irrawaddy and Chindwin Rivers would not only depend on the manner of its operation but also on the physical characteristics of the streams. The width and depth of the channels and particularly the degrees of curvature of the bends would be the critical factors in determining the size and length of push towboat and barge units. Before investing heavily in push towboats, I.W.T.B. plans to conduct experiments with trial tows using available plant and converted equipment to determine whether push towing of units large enough for economical operation can be used on the river.

Should the riverine commerce continue as small lots of varied general cargo and material passenger trade, then the prewar type of vessel probably will be the more advantageous; however, should the heavy bulk commerce of coal movement from Kalewa on the Chindwin River and petroleum from the Chauk area on the Irrawaddy develop, then there seems little doubt that the advantages in maneuverability and economics of push towing will prescribe that type of operation for the new trade.

## Private Operations

According to available data there are at least 180 registered privately owned riverine craft operating on the Irrawaddy River and Delta System and in the Moulmein area in competition with the Inland Water Transport Board. These private vessels represent more than 30 per cent of the number of I.W.T.B. craft, and are mostly motor launches with a maximum capacity of 120 tons. More than 70 per cent of them operate in the Irrawaddy Delta. The owners of these craft are organized in an association known as "The Burma Inland Water Transport Union Headquarters."

## Dockyards

The Inland Water Transport Board operate their own vessel repair units. The principal dockyard, known as the Dalla Dockyard, is located on the right or south bank of the Rangoon River opposite the City of Rangoon. This facility covers an area of 52 acres and has ample space for expansion. The dockyard is well equipped with shops, thirteen shipways, and one wet dock. The capacity of the shipways varies from one 50-ton craft to two 500-ton craft. Shipways Nos. 7 to 12 have six carriages capable of taking 200 tons each; they are designed to haul sideways a ship of 1,200 tons up to 250 feet in length. The wet dock has a length of keel blocks of 220 feet and a width at entrance of 47 feet 3 inches at the seven-foot tide mark. It is capable of taking craft up to 230 feet in length of approximately 1,000 gross tons.

The Rangoon Foundry on the left bank of the Rangoon River is a subsidiary of the Dalla Dockyard. It has a foundry, machine shops, plating and copper shops, and four shipways, two of which are 100 feet long and two 80 feet long. The shipways can accommodate two launches 94.5 feet long and three cargo boats up to 450 tons each.

The Inland Water Transport Board also maintains marine repair dockyards at the river port of Mandalay and at the seaport of Moulmein. For the vessels operated in the Arakan area the craft are repaired in the dockyard maintained by the Arakan Flotilla Company.

The gradual rebuilding of the general cargo and passenger fleet and the continued shift toward diesel power will impose a requirement for the additional training of personnel. Foundry, galvanizing plant, and diesel shop supervisors, diesel engine operators, and marine structural and mechanical designers will be required. If barge trains propelled by push towboats are adopted, pilot training schools and training tow units also will be needed.

## Conservancy

River conservancy on the Irrawaddy and Chindwin Rivers is the responsibility of the Mercantile Marine Department of the Ministry of Transport and Communications, but certain conservancy work is carried out by the Department of Irrigation in their reclamation and flood control operations in the Delta area and by I.W.T.B. in their navigating operations. Channel maintenance and marking is primarily the function of the Mercantile Marine Department, but the I.W.T.B. also has some plant engaged in marking channels. The Department of Irrigation maintains the Twante Canal, a 21-mile artificial channel connecting the Port of Rangoon with one of the main distributaries of the Irrawaddy River.

The conservancy fleet and marine plant of the Mercantile Marine Department suffered severe damage during the war. Consequently, whenever necessary, the Department of Irrigation, which operates two dredges and other marine equipment, carries out channel maintenance work particularly in the Irrawaddy Delta at the request of I.W.T.B. and in agreement with the Mercantile Marine Department. On the upper Irrawaddy and the Chindwin Rivers there are at present in operation two snag boats and a rock pounder. One salvaged dredge is being reconstructed and rehabilitated by the Mercantile Marine Department in its Mandalay boatyard. Three additional dredges are needed for channel maintenance on the Irrawaddy System and in the Moulmein

and Arakan areas. Six snag lifters and three rock pounders are required for the Irrawaddy and Chindwin Rivers. The Mercantile Marine Department operates the Dawbon Boatyard which is located on the left bank of Pazundaung Creek opposite the City of Rangoon and which is engaged principally in the construction and maintenance of the equipment used for conservancy in the Delta area and for the conservancy of the entrance channels to Burma's outports.

The presently adopted project depths and widths of the navigated channels of the Irrawaddy River system are as follows:

| | Minimum Depths | Minimum Widths |
|---|---|---|
| Rangoon to Mandalay (Irrawaddy R.) | 4'-6" | 150' |
| Mandalay to Bhamo ( " ) | 4'-0" | 100' |
| Pakokku to Mawlaik (Chindwin R. ) | 3'-6" | 100' |
| Mawlaik to Homalin ( " ) | 2'-9" | 100' |
| All Delta Waterways | 5'-6" | 150' |

It is probable that a minimum depth of 5'-3" can be economically maintained between Rangoon and Mandalay.

The existing project dimensions are adequate only for the prewar type of general cargo and passenger vessels. The limits would be inadequate for safe and dependable operation of the fleets required for the proposed Kalewa coal development, the proposed industrial projects in the vicinity of Myingyan, and for the riverine transport of petroleum from the producing fields in the Chauk area to Syriam in the Rangoon area where the prewar oil refineries were located. To provide adequate channels for the fleets required to transport the products of these developments, project dimensions will have to be increased as shown below:

| | EXISTING | | PROPOSED | |
|---|---|---|---|---|
| | Depth ft. | Width ft. | Depth ft. | Width ft. |
| Irrawaddy River between Rangoon and Myingyan | 4.5 | 150 | 5.5 | 190 |
| Irrawaddy River between Myingyan and Mandalay | 4.5 | 150 | 5.0 | 150 |
| Chindwin River between Pakokku and Kalewa | 3.5 | 100 | 4.0 | 130 |

The channels would have to be widened on the inner side of all bends; the extent of widening would be determined by the degree of curvature of each bend.

## Economic Considerations

In the Rangoon — Mandalay trade, the principal flow of goods in Burma, transport can be accomplished either by river, railway, or highway. In

general, security conditions are similar, yet there is a difference in the costs of transport. The river haul is 552 miles and the rail haul is 385 miles. The following tabulation shows a comparison of hauling a few of the goods by water and rail.

| | I.W.T.B. | | BURMA RYS. | |
|---|---|---|---|---|
| | Per Ton | Per Ton-mile | Per Ton | Per Ton-mile |
| Rice | $16.03 | $.0291 | $ 9.02 | $.0234 |
| Grains and Pulses | 9.10 | .0165 | 9.02 | .0234 |
| Sugar | 16.03 | .0291 | 12.34 | .0321 |
| Salt | 13.58 | .0246 | 9.02 | .0234 |

The ton-mile rate for hauling cement from Thayetmyo (276 miles above Rangoon) by water and where there is no rail competition is 3.00 cents. Water rates for the transportation of petroleum products from Chauk to Rangoon range from 2.62 cents per ton-mile for kerosene to 6.00 cents per ton-mile for gasoline. Chauk is not served by rail although a branch line of Burma Railways extends to within 22 miles of the town.

Both agencies are government-owned, and no rate regulating board has been established since both the waterways and railways are administered by the Ministry of Transport and Communications, which ministry has the responsibility of coordinating the activities of all transportation means except highway transport. In the case of the private and contract carriers and the commerce engaged in by the country craft it is not possible to exercise enough control to fix the rates, and therefore the rates for transport by those craft are more often set by barter. There will be an urgent need for consistent transportation rates and services as the Union of Burma develops.

Development of new projects under consideration by the Government of the Union of Burma will increase the traffic for all modes of transportation. The largest near-future increases in riverine commerce will come from the Kalewa coal development. By 1959-1960 it is estimated that approximately 370,000 tons of washed coal, briquetted coal, and metallurgical char and briquettes will move down the Chindwin River annually for distribution to the Myingyan, Mandalay, and Rangoon areas. There will be no return cargo of consequence in the foreseeable future. The number of barges and towboats, with spares, required in the coal trade for the various transport sections, is estimated as follows:

| | Barges | Towboats |
|---|---|---|
| Kalewa to Mandalay | 10 | 5 |
| Kalewa to Myingyan | 24 | 13 |
| Myingyan to Rangoon | 22 | 6 |
| | 56 | 24 |

It has been assumed in estimating the fleet and transportation costs that twin-screw push type towboats would be found suitable for use on both the Irrawaddy and Chindwin Rivers. This assumption must be verified by trial

tows using available plant and converted equipment as previously mentioned. The river channels would require widening and deepening to the dimensions proposed in the aforementioned tabulation of channel dimensions to permit safe, dependable, and economical transportation. The cost of the transportation (exclusive of channel conservancy costs) of Kalewa coal and coal products from the mine to the several market centers is estimated to be:

| | |
|---|---|
| From Kalewa to Mandalay | 1.44 cents per ton-mile |
| From Kalewa to Myingyan | 1.50 cents per ton-mile |
| From Myingyan to Rangoon | 0.96 cents per ton-mile |

The revival of the production in the oil fields that skirt the Irrawaddy River in the vicinity of Chauk about 410 miles upriver from Rangoon may provide additional commerce for the waterways. Before the war crude oil was pumped from the oil fields to the refineries at Syriam near Rangoon; however, military activities destroyed at least one-half of the pipeline and all of the line stream crossings and pumping stations. Considerable expenditure will be required to rehabilitate the pipeline system. The refineries at Syriam were destroyed completely during the war, but a small refinery of 2,500 barrels per day capacity was completed recently at Chauk. The products of this refinery move by river, but the rates are high and service uncertain.

It is estimated that the annual requirement for petroleum products in Burma will approximate 98,000,000 gallons by 1959-1960, or a production of 8,000 barrels per day. Comparative analyses indicate that water transportation of oil to refineries rehabilitated at Syriam can be accomplished by a fleet of barges at rates less than required in the rehabilitation of the pipeline.

Water transportation on the Irrawaddy River system will continue to be a major factor in the national economy of Burma and will remain the principal means of transport in the Delta and valley; however, as in all other areas where waterways are the means of commerce, there are the attending problems.

## CONCLUSIONS

In the 10,000 square miles of Delta area with over 2,000 miles of commercially navigable waterways the transport problems are clear and easily controlled. Here, water transport will remain the only dependable and economical means because of the great expense involved in the construction of railways and roads through this low area with its intricate maze of rivers, creeks, and waterways. The fast and highly maneuverable delta launch now operating in this area should remain the most suitable craft. With that type of vessel conservancy problems are relatively few, for the silt load and drift debris are concentrated in the principal distributary rivers, of which there are eight, and are carried beyond to the Andaman Sea. The main problem is the operation of an adequate number of craft to furnish the scheduled services needed for the commerce. For transporting the heavy seasonal trade in rice paddy it would be impractical to replace the cheap country craft and lighters with motorized craft.

On the main Irrawaddy and Chindwin Rivers navigation will continue to be controlled by natural conditions and by the demands of commerce. The low

water channels which prevail for not less than seven months of each year are highly meandering and heavily beset with sand bars, and it is the degree of meandering and depths over bars which control the dimensions of the craft. Probably the best and most economical method for obtaining better navigating conditions would be through the use of relatively reasonable river training works such as deflecting dikes, bank stabilization, and other low-cost works; however, until the commerce increases materially heavy expenditures for increasing the project dimensions of the channels greatly could not be justified economically.

As long as general cargo and passenger traffic control the type of operating craft, the need will be for fast movement of the commerce in comparatively small lots and scheduled operations of high frequency. It will be only when large tonnages of bulk cargo are handled, such as the movement of petroleum and the Kalewa coal and coal products, that the great economy in water transport can be realized, when fleets of barges can be propelled by push towboats, especially when the principal traffic movement is downbound. Under those provisions the necessary channel conservancy operations could be justified economically to materially better navigation and riverine transport conditions.

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# MODEL STUDIES OF SECTOR GATE TYPE LOCKS

Frederick R. Brown,[1] M. ASCE
(Proc. Paper 1767)

## INTRODUCTION

Once the size of a lock has been determined, the next important decision concerns the type of filling system that will be used. From the many types of systems, and many combinations of types that are known, the system is usually selected on the basis of the following considerations:

a. Economy of construction and operation.
b. Length of filling time.
c. Magnitude of dynamic forces acting upon tows or vessels within the lock chamber.

This paper discusses a few of the model studies conducted at the Waterways Experiment Station on sector-gate-type locks, and in particular those studies conducted on a model of the Calumet Lock. The use of the sector-gate type of system is not so widespread as the familiar miter type and culvert control, although it is not a new system and, in fact, has been used in this country and Europe for some time, particularly where reverse head conditions might be encountered. Some of the earliest information available on the design and operation of sector lock gates came from Sweden.

Sector gates are in use in the St. Lucie and Ortona Locks in Florida; in the Algiers Lock in Louisiana; and are incorporated in final plans for the Sacramento Barge Canal Locks in California. Because of their ability to be opened or closed under head conditions, sector gates have been used for control of flow through the Chicago Drainage Canal in Illinois, at Lake Okeechobee in Florida, and at various places on the Gulf-Intracoastal Waterway System in Louisiana and Texas.

The sector-type gate rotates in a horizontal plane about a vertical pintle and, as the name implies, is the sector of a circle with a circular skin plate.

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1. Chf., Hydrodynamics Branch, U. S. Army Engr. Waterways Experiment Station, Vicksburg, Miss.

Two gates provide a barrier to flow by joining at the longitudinal axis of the lock. Access into or out of the lock is secured by swinging each gate into large recesses in each lock wall.

In general the highest water level occurs against the convex side of the skin plate which causes the resultant of all pressures to pass through the center of rotation of the gate pintle and place gate members in compression. For reverse head conditions, with highest pressures on the concave side of the skin plate, the gate members are in tension. As the sector gates are opened flow may pass into the lock chamber through the central opening only, or it may pass both through the central opening and around the sides of the gates. Specific examples of each condition will be given in later paragraphs.

Economic considerations of the sector gate reveal that the two gate leaves are generally more expensive to construct than a single set of miter gates. However, under conditions of reverse head a single set of sector gates is less costly than a double set of miter gates and a complicated culvert and lateral or port system of filling. Another advantage is that the gates can be operated under head thus permitting the lock to be used for passage of drift or scour of the upper and lower lock approaches if desired. Actuating power requirements are small.

Disadvantages of sector gates are the longer filling time required to secure acceptable hydraulic conditions within the lock chamber. For head in excess of 20-25 ft., the time of filling is so long that some other type of system is usually considered.

Since the lock chamber is filled by water flowing in at one end, it is obvious that large longitudinal forces are created within the lock, accompanied by turbulence in the upper end, which are hazardous to barge tows or vessels in the chamber. The magnitude of the longitudinal forces and degree of turbulence are directly related to the rate of inflow, and result in the head limitation cited above. The sealing of the sector gates is another problem in their use.

Before the lock at Algiers was constructed the maximum head differential under which sector gates had been used was about 6 ft. Algiers Lock is designed to fill at a maximum head of 18.5 ft. and the Sacramento Lock, currently under investigation at the Waterways Experiment Station, will have a maximum head of 21.1 ft. The Calumet Lock has a maximum head of 9 ft but the lock chamber is considerably wider than those of the other two mentioned.

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## Algiers Lock

### Description

Algiers Lock is located on the west bank of the Mississippi River, opposite and a short distance south of New Orleans. This lock has a usable length of 760 ft, a width of 75 ft, and a lift from the Intracoastal Waterway into the Mississippi River of 0 to 18.5 ft. At high tide and low river stages it is possible for a reversal of head to exist, requiring a lift of 4.5 ft from the river to the waterway. Filling and emptying of the lock chamber are accomplished by partially opening the gates and allowing the water to flow through the opening between the gates and also around each gate between the skin plate and the gate recess. The streams of water entering from the sides impinge on

each other, which tends to dissipate the energy contained in each as well as that in the stream of water entering from the center, thereby reducing the turbulence in the upstream end of the lock chamber.

## Model Tests

A 1:20-scale model was used to determine a method of gate operation that would permit filling as rapidly as possible without endangering tows moored within the lock chamber. Model construction and operation procedures were similar to those used for tests of the Calumet Lock to be described later.

As a result of the model tests a schedule of variable rates of gate opening was developed which reduced longitudinal forces on barge tows to about 12 tons for conditions of maximum head with 8 barges in the lock. The time of filling under a head of 18.4 ft was 9 to 10 minutes. Alterations to the side seals of the sector gates improved locking conditions by permitting more flow to pass around the sides of the gates. These alterations also resulted in a more balanced force condition acting on each of the seals. Initially a large closing force was noted which was caused by a drawdown in the water surface past the gate lip in the recess (Fig. 1) while a level water surface acted against the projected area of the gate lip in the center of the lock passage. Realignment of the gate recesses reduced impact of side flow on the gate members and pintles. Detailed model results have been published in an Experiment Station technical report and will not be repeated in this paper.

Satisfactory filling was contingent upon the speed of operation of the sector gates. Several methods of operating the gates at variable speed were considered, and it finally was decided that the best results would be obtained with hydraulically-operated struts accelerated by electrically-driven pumps. For flexibility and speed of operation, each gate was provided with two types of pumps: a variable displacement pump for slow speeds, and a vane type pump for high speeds. Gate machinery is controlled by valves, relays, limit switches, and interlocks so as to provide semi-automatic operation.

## Prototype Performance

Since construction the Algiers Lock has performed about as expected although the maximum head has not yet been experienced. Immediately after construction and before the lock was opened to traffic observations for a 14-ft head were made. It was noted at that time that leakage past the side seals and through the bottom seals set up vibration in the various component parts of the gate. Actual flexure of some of the longer gate members was noted and a fatigue failure was feared. As the gates were operated the noise of the vibration reached its highest pitch at about a 3-in. gate opening and stopped after the gates were opened about 2 ft. Realignment of the seals eliminated the vibration with the gates in the closed position.

Although efforts had been made during the course of the model study to balance the forces acting on the sector gates, it was observed under prototype conditions that the gates tended to creep open from the closed position. Thus, a closing force had to be maintained at all times by the operating machinery. This indicates that in future designs a slightly unbalanced condition with the closing force predominant should be obtained to insure sealing when the gates are closed.

As observed in the model, the area in the lock chamber about 40 ft downstream from the gate pintles was very turbulent and probably would be harmful to tows moored in the area.

Figs. 2-8 show the leakage past the gates and flow conditions in the lock chamber during filling.

## Sacramento Lock

### Description

Another example of a sector-gate type lock is the Sacramento Barge Canal Lock presently under study at the Experiment Station on a 1:20-scale model (Fig. 9). This lock will be 86 ft wide with an over-all length of 640 ft (usable length 600 ft) and will be located in a barge canal connecting Washington Lake to the Sacramento River. Lake Washington will be joined to the ocean with a deep-water ship canal. Maximum lift is 21.1 ft. Design criteria call for filling the lock chamber in 12 minutes under a 12-ft head. The reverse head is estimated at 3.6 ft and will occur at infrequent intervals. As in the case of Algiers Lock, flow will pass into or out of the lock both between and around the sector gates.

The gates have a sector angle of 74 degrees and 56 minutes with a radius to the inside of the skin plate of 43 ft, 6-1/4 in. The gates are to be operated through a rack located at the upper edge of the skin plate with power supplied by electric motors acting through a speed reduction unit and system of gears.

Since the general dimensions of the Algiers and Sacramento Locks were somewhat similar, the Sacramento design was patterned after the Algiers design in so far as was practicable. However, it was desired to move the gates at a constant rate instead of the variable rate developed for Algiers Lock.

### Model Tests

Initial considerations called for investigations of several rates of gate opening in an effort to secure the desired filling and emptying characteristics. Opening the gates at the constant angular rate of 0.24 degrees per minute resulted in a filling time of 17.5 minutes. Therefore subsequent tests were conducted with faster rates of gate opening. After consideration of a number of rates of opening, the following schedule was adopted for filling the lock chamber.

Filling Schedule

| Head Differential (Ft) | Rate of Sector Gate Opening (Deg/Min) | Filling Time (Min) | Maximum Hawser Stress (Ton) |
|---|---|---|---|
| 0 - 6 | 0.66 until lock fills | 9.6 | 2.7 |
| 6 - 12 | 0.33 for 6 min then 0.66 until lock fills | 12.6 | 5.3 |
| 12 - 21 | 0.33 for 2.5 min, stop 10 min, then 0.33 until lock fills | 22.8 | 6.2 |

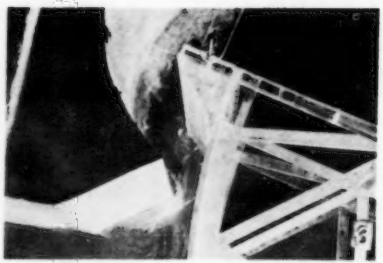


Fig. 1. Flow past side seal, model gate

Fig. 2. Prototype view, upper gate

Fig. 3. Prototype view, lower gate

Fig. 4. Left
Leakage at side seals

Fig. 5. Right

Fig. 6. Flow through center seals, 14-ft head

Fig. 7

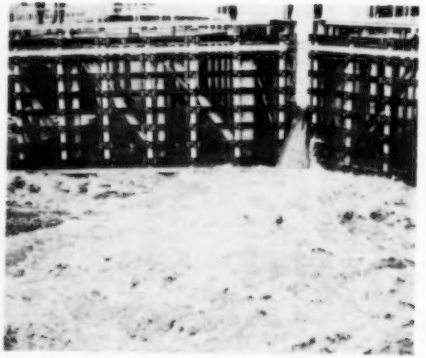


Fig. 8
Turbulence in lock chamber at upper gates during filling

MODEL AND PROTOTYPE VIEWS, ALGIERS LOCK

For a 12-ft head (normal) the lock could be filled in about 12.6 minutes with maximum hawser stresses for tows within the lock chamber in the range of 5.3 tons. For a head of 21 ft, a rare condition, a filling time of 22.8 minutes was required to keep hawser stresses near the desired limit.

For emptying the lock chamber, a rate of gate opening of 0.66 degrees per minute for all heads was satisfactory. For heads of 6, 12, and 21 ft the time of emptying was 10.4, 12.7, and 12.8, respectively; maximum hawser stresses did not exceed 3.2 tons. The almost identical emptying time for 12- and 21-ft heads is the result of the lesser depth of water over the sill at the lower head.

Forces required to operate the gates also were determined, but in view of the experiences at Algiers Lock, a slightly greater closing force was maintained at all times. Other tests on the model were concerned with the magnitude of velocities and distribution of flow in the downstream lock approach.

## Calumet Lock

### Description

The Calumet River Lock in South Chicago, Illinois, will be part of a project for widening and deepening the Calumet-Sag Channel, an important link in the Great-Lakes-to-the Gulf Waterway. The lock will be 1000 ft long, 110 ft wide, and will provide a depth of 12 ft over the sills. Actually a lock length of 950 ft would be sufficient, but on the basis of the previously described tests of Algiers Lock, 1000 ft was considered desirable so that tows would not have to be moored in the turbulent area immediately downstream from the gates.

As indicated by previous papers, the fluctuations in lake levels and the river stages that occur during flood periods indicate the use of sector type gates to withstand a head of water in either direction. It was estimated at Calumet Lock that the normal operating head would range from 0 to 5 ft with a maximum differential head of 9 ft possible on either side of the gate. Actual water surface in the channel will range from an elevation of -2 to +4*, whereas the lake level will vary from +5 to -4.9 (maximum recorded range). Normal operation will require lifts from the river channel to lake elevation. Actual elevation of the lock floor will be -18.5 with the sill at elevation -17.

It was originally planned that the gates for the Calumet Lock would be similar to those used at Algiers Lock, to minimize filling time. However, because of ice conditions, it later was decided to fill and empty only between the sector gates and, if necessary, to supplement filling by means of loop culverts.

### Requirement for Model Studies

Model studies of the Calumet Lock were considered desirable because of the large width (110 ft) and the fact that filling was to be accomplished through the center opening only. It was also desired to determine the speed of gate operation that would produce acceptable hydraulic conditions within the lock for all conditions of head. Design criteria called for filling the lock in about 8 minutes with a lake level of +3 and a river stage of -2 (normal condition of head). Hawser stresses on moored tows were not to exeed 5 tons.

* Elevations are in ft referred to Chicago City Datum.

The feasibility of decreasing the filling time by use of loop culverts discharging either into the gate recesses or through the lock wall or floor downstream from the recesses also was to be determined. These culverts were to be considered only in the vicinity of the lakeward gates as the reversal of head would be too infrequent to warrant consideration in the riverward gates.

## Description of Model

A 1:20-scale model (Fig. 10) reproducing the entire lock and immediate approaches was constructed. Provisions were made for mechanical operation of the sector gates and automatic recording of pertinent data. One set of sector gates (Fig. 11) was fabricated of plastic and placed in the end of the lock chamber under investigation. The gate recesses were molded of concrete to sheet metal forms shaped to the desired curvature. The loop culverts were fabricated of sheet metal with vertical-lift control gates. Barges also were fabricated of sheet metal and assembled into tows of the desired length; each barge simulated a length of 195 ft, a width of 32 ft, and was loaded with sand to the desired draft.

A skimming weir located in the headbay area of the model maintained a constant water-surface elevation; a tailgate at the downstream end of the model served the same purpose during emptying and head reversal tests. The sector gates were actuated by electrically driven cams, the horizontal movement of which opened the gates in a predetermined time interval. The cams were of plywood and easily fabricated to permit tests of many speeds of gate operation. Loop culvert gates were operated by means of cables over sheaves to a manually-operated winch.

Pressure cells and wave rods were used to measure the water-surface elevations in the lock and in the upstream and downstream areas. A potentiometer was used to record position of the gate leaves. Hawser stresses, longitudinal and transverse, on the barge tows were measured by means of semicircular aluminum links to which SR-4 strain gages had been cemented (Fig. 11a). One end of the link was pin-connected to the tow while the other end was fastened to a vertical rod fixed in the chamber. The link was free to move up and down the vertical rod as the water-surface elevation in the lock chamber was varied. The method of mooring tows in the lock chamber is not representative of full-scale operations. It does provide an answer, however, as to the relative effect of the speed of gate operation, head, arrangement of barges and draft on the forces required to hold a tow in position. All of the information was recorded in graphic form on a paper chart (Fig. 12).

## Test Program

For the most part tests were conducted with a single tow of 8 barges arranged 2 abreast and loaded to a draft of 9 ft. Thus, each tow had an overall width of 64 ft and length of 780 ft. For an end filling system the full size tow should provide the most critical hawser stresses. The tow was moored on the center line of the lock, 50 ft from the upstream gate pintles. Tests were usually conducted with the normal head of 5 ft procured by maintaining an upper pool of +3 and a lower pool of -2. Variations in these test conditions were made during the course of the study. Also the program was reviewed and results discussed at frequent intervals as illustrated by Fig. 13 which shows representatives of the Chicago District and North Central Division of the Corps of Engineers, and the Illinois Central Railroad.

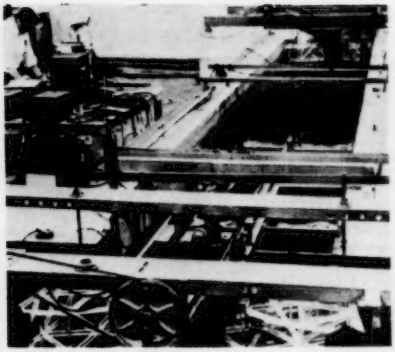


Fig. 9. Model of Sacramento Barge Canal Lock; scale 1:20

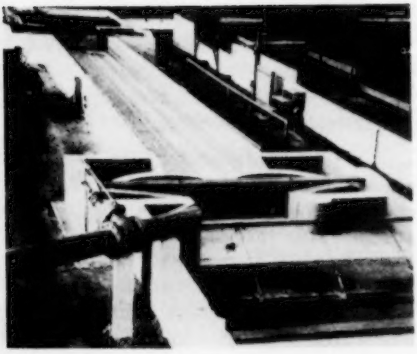


Fig. 10. Model of Calumet River Lock; scale 1:20

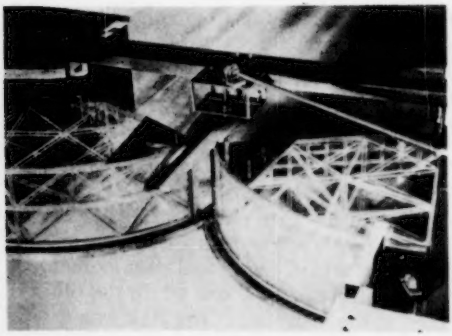


Fig. 11. Model of Calumet sector gates

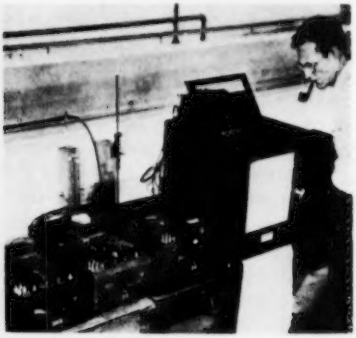


Fig. 12. Recording equipment

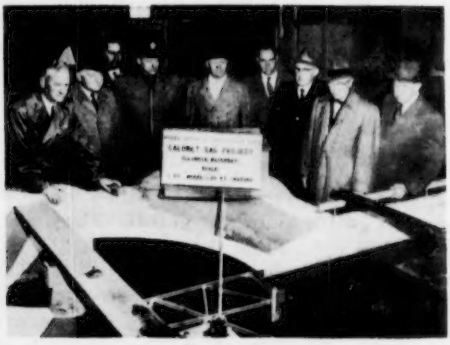


Fig. 13. Conference on model results

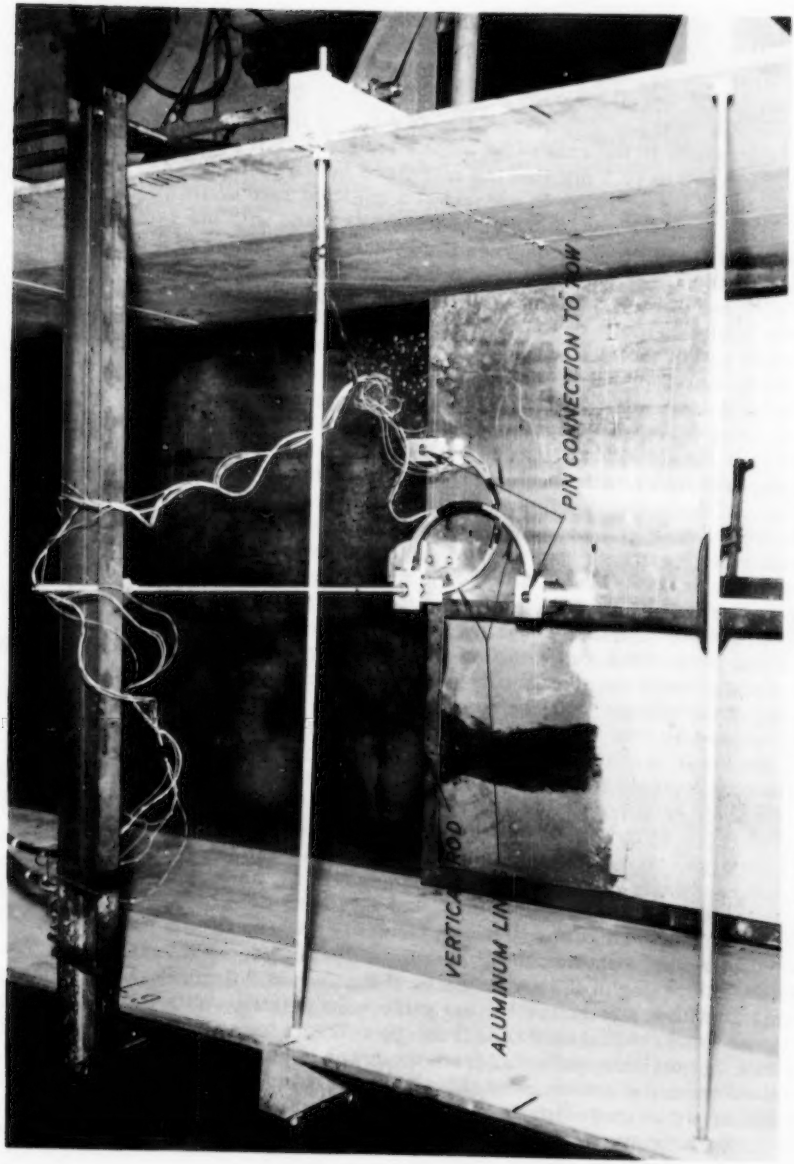



FIG. 11a. INSTRUMENTATION FOR MEASUREMENT OF HAWSER STRESSES.

Filling Tests

Initial tests were concerned with flow between the sector gates only. These tests revealed that opening the gates rapidly to some predetermined opening for a specified time was unsatisfactory. Hawser stresses were considerably in excess of allowable limits (5 tons). Tests also were made with the center opening varied at a constant rate. For example, opening at the rate of 0.75 ft per minute resulted in a filling time of 14.7 minutes and hawser stresses in the range of 7 tons; a rate of 1.25 ft per minute reduced the filling time to 10.7 minutes but increased the hawser stresses to nearly 9 tons. Since the increase in opening at a constant rate would require a variable drive for the gates, a series of tests was made with the gates opened at a constant rate of so many degrees per minute. An opening of 2 degrees per minute resulted in a filling time of 8.5 minutes and hawser stresses as large as 10.4 tons (Fig. 14). In all tests it was noted that as the gates were opened the tow tended to move downstream because of the increase in water level at the upper end of the lock chamber. However, within a short time the tow started to move upstream because of the reflection of the surge wave from the downstream set of gates. The resultant force upstream was usually larger and of longer duration than the downstream force.

Since it did not appear that the desired time of filling within the allowable hawser stresses could be attained by filling through the sector gates alone, the use of loop culverts was investigated. Initial tests were made with 8-ft-diameter culverts positioned with the intakes approximately 15 ft upstream from the gate bays and discharging into the recesses so that flow from the culverts would not impinge directly on tows located within the lock chamber. Also it was desired that the inflow into the upper end of the lock chamber be more uniformly distributed and be parallel to the lock walls.

Filling tests conducted with flow from the culverts only indicated that about 22.5 minutes were required to fill under a 9-ft head regardless of the rate of opening of the vertical-lift control gates. It also was noted that the surge wave created by flow into the upper end of the lock chamber was reflected from the lower set of gates within about 1 to 1.5 minutes.

The next logical step in the investigation was the development of a schedule for combined culvert and sector gate operation. Synchronous and non-synchronous operation of the culverts and sector gates were investigated. For these tests the upper pool was maintained at elevation +3 and the lower pool at -2 (normal conditions). The test results indicated that, as expected, the use of both culverts and gates resulted in a shorter filling time. Non-synchronous operation of the culvert and sector gates also appeared to be advantageous. Best results were obtained by opening the culvert gates at the rate of 4 ft per minute with opening of the sector gates at the rate of 2 degrees per minute begun 1.25 minutes after the culvert gates were started. With this schedule the surge wave created by inflow from the culverts traveled down the lock chamber, but on being reflected from the lower set of gates it was damped by the inflow from the sector gates, thus reducing over-all hawser stresses, particularly in an upstream direction. As pointed out previously this is the most critical direction. For this condition the time of filling was 8.56 minutes and the maximum hawser stress 4.3 tons in an upstream direction.

Several other types of culverts were investigated wherein flow was introducted into the lock downstream from the gate recesses and through the lock floor. Best results, however, were obtained with flow from the culverts

introduced into the lock recesses since much of the turbulence was dispersed before the flow entered the lock chamber.

Use of a 10- by 10-ft square culvert instead of the 8-ft-diameter culvert increased the cross-sectional area by a factor of 2 and permitted a faster filling time. A series of tests conducted with this culvert revealed optimum conditions with the culvert gate opened at the rate of 2.5 ft per minute, a time lag of 1.25 minutes, after which the sector gates were opened at the rate of 2 degrees per minute (Fig. 15). With a 5-ft lift the lock was filled in 6.45 minutes; maximum hawser stresses were 4.8 tons in an upstream direction and 4.0 tons in a downstream direction. On the basis of additional studies the following operating schedule for filling the lock chamber was recommended.

| Head Differential (Ft) | Culvert Gate (Ft/Min) | Sector Gate | |
|---|---|---|---|
| | | Rate (°/Min) | Lag Time (Min) |
| 0 - 3 | 2.5 | 2 | 0 |
| 3 - 6 | 2.5 | 2 | 1.25 |
| 6 - 9 | 2.5 | 2 | 3.50 |

When the head is within about 0.25 to 0.50 ft of equalization, the gates can be opened as fast as 30 degrees per minute.

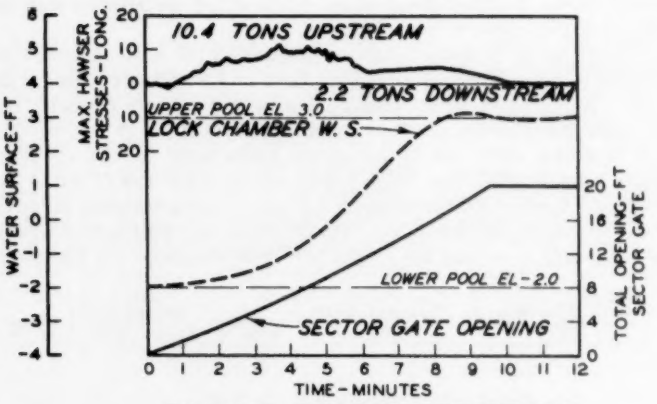
Hawser stresses for each of the above conditions will vary somewhat, depending upon the draft and location of the barge tows and upon the water cushion within the lock chamber. An effort to determine this variation revealed about 1.5 tons more hawser stress for equivalent heads with minimum water level (-4) in the lock chamber (Fig. 16). This is attributed to the additional friction drag on the bottom of the tow. The actual time of filling the lock chamber also was contingent upon the depth of water in the lock chamber. The shallower the depth of water under a tow the longer the time of filling.

### Effect of Barge Tow

Draft. Drafts of 1.6, 5.0, and 9.0 were investigated with an 8-barge tow and normal head conditions (+3 upper pool, -2 lower pool). The resulting hawser stresses measured indicated that, as expected, the forces decreased almost directly with draft. For a tow located 100 ft downstream from the gate pintle the hawser stress varied from 4 tons at 9 ft draft to 1 ton at 1.6 ft draft (Fig. 17).

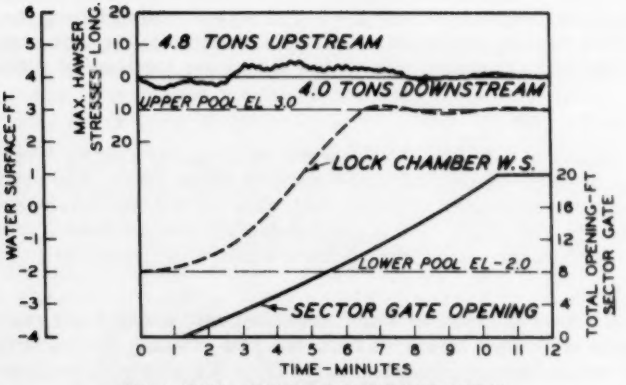
Location. Hawser stresses were measured with tow of 8 barges at 9-ft draft moored at various distances from the gate pintles. With a normal lift of 5 ft, maximum hawser stresses varied from 3.2 tons with the lead barge 10 ft from the gate pintles to 5.0 tons with the lead barge 75 ft from the pintles. The hawser stresses decreased farther the tow was moored from the pintles (Fig. 18). It is obvious, however, that tows should be moored outside of the turbulent area immediately downstream from the gates to avoid possible contact with the gates during the filling operation. The hawser stresses for a tow moored any place in the lock chamber were within allowable limits for a 5-ft lift. For heads as great as 9 ft the hawser stress could reach 6.4 tons.

Arrangement. Tests conducted under normal head conditions with a four-barge tow indicated that longitudinal hawser stresses were of about the same



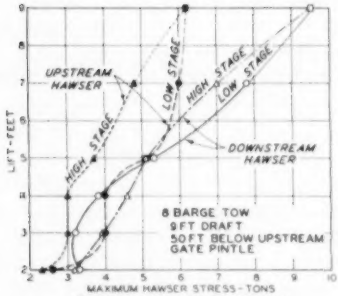

FILLING CHARACTERISTICS
SECTOR GATE OPENING 2 DEG/MIN

Fig. 14



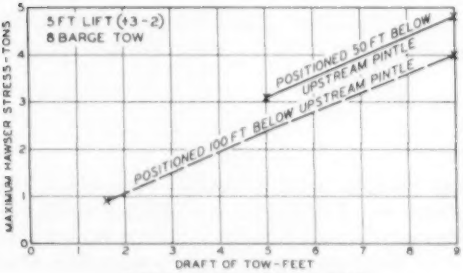

FILLING CHARACTERISTICS
2.5 FT/MIN CULVERT 2 DEG/MIN SECTOR GATES
1.25 MIN LAG

Fig. 15



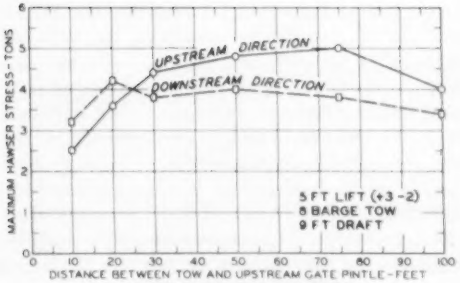

EFFECT OF WATER DEPTH ON HAWSER STRESSES

Fig. 16


5 FT LIFT (+3-2)
8 BARGE TOW
POSITIONED 50 FT BELOW UPSTREAM PINTLE
POSITIONED 100 FT BELOW UPSTREAM PINTLE
MAXIMUM HAWSER STRESS-TONS
DRAFT OF TOW-FEET

EFFECT OF TOW DRAFT ON HAWSER STRESSES

Fig. 17




EFFECT OF TOW POSITION ON HAWSER STRESSES

Fig. 18

EFFECT ON BARGE TOWS

magnitude (3.2 tons) whether the tow was arranged two barges abreast, or four barges in train. For the latter condition transverse hawser stresses were greater than the longitudinal stresses. For an 8-barge tow and the same conditions of head and gate operation, hawser stresses were about 4.8 tons.

Emptying Tests

Test results indicated that, with +3.0 lock chamber and -2.0 lower pool elevation, satisfactory conditions could be obtained by operation of the sector gates at a constant angular velocity of 5 degrees per minute until the lock chamber was empty. This procedure resulted in an emptying time of about 6.4 minutes with hawser stresses less than 5 tons (Fig. 19). The tests were conducted with an 8-barge tow at 9-ft draft located 50 ft upstream of the skin plate of the lower sector gates. The following schedule of gate operation was recommended for the lock emptying procedure.

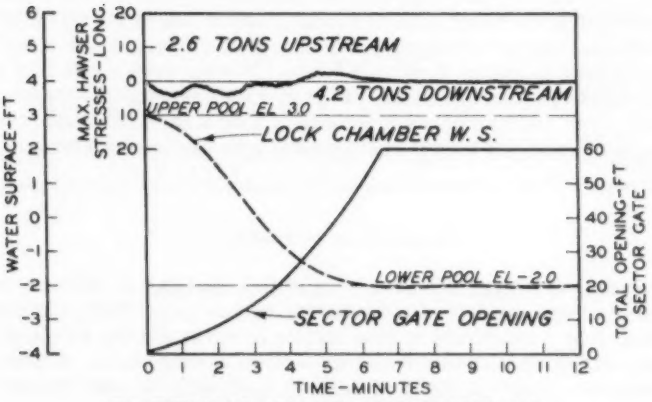
| Head Differential (Ft) | Sector Gate Deg/Min |
|---|---|
| 0 - 4 | 6 |
| 4 - 9 | 5 |

Effect of Head Reversal

Filling tests under reverse head conditions were made with the lock chamber maintained at elevation -4 while the downstream level was varied from -1 to -3. Tests indicated that the hawser stresses varied almost as a straight line for the first 3 ft of head and ranged from a minimum of about 3 tons at a 1-ft head to 8.6 tons at a 3-ft head. The maximum force was always in a downstream direction. The time of filling at a 2-ft head reversal (normal reverse head) was 4.7 minutes for a gate operated at 5 degrees per minute and 4.4 for a gate operated at 6 degrees per minute. Typical filling curves for a 2-ft head are shown on Fig. 20.

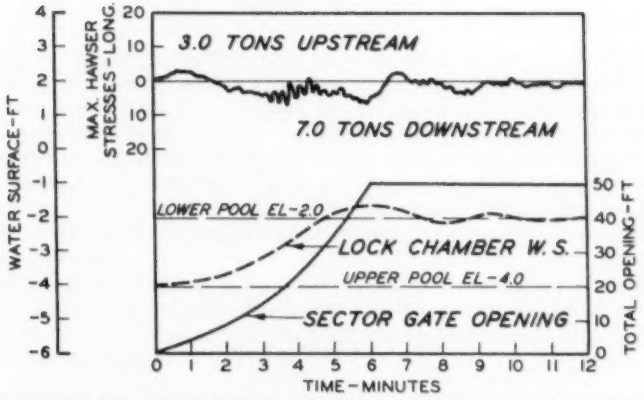
Since the hawser stresses exceeded 5 tons when the lower set of gates was operated at the speed recommended for emptying the lock, tests were conducted with the gates opened at the rate of 2 degrees per minute (same as lakeside gates) for a definite period of time and then stopped until equalization of head had been accomplished. Hawser stresses were reduced appreciably. For example, under a 9-ft reverse head, if the gates were stopped after 2 minutes of operation, the maximum hawser stress would be reduced to 5 tons; the time of filling would be 26.5 minutes. For a 7-ft head the hawser stresses did not exceed 5 tons if the gate movement were stopped after 2.5 minutes; time of filling was 21.8 minutes. If it is necessary to maintain the limit of 5 tons for hawser stresses, the following schedule for filling under reverse head conditions is recommended.

| Head Differential (Ft) | Sector Gate Opening (Deg/Min) | Time Leaf Opened before Stopped to Permit Lock to Fill (Min) |
|---|---|---|
| 0 - 1 | 2 | Until filled |
| 1 - 7 | 2 | 2.5 |
| 7 - 9 | 2 | 2.0 |




EMPTYING CHARACTERISTICS
SECTOR GATE OPENING 5 DEG/MIN

Fig. 19




REVERSE HEAD FILLING CHARACTERISTICS
SECTOR GATE OPENING 5 DEG/MIN

Fig. 20

If, however, the cost of providing another operating speed for the lower set of gates is important, consideration should be given to use of the same operating speeds developed for the emptying procedure (5 to 6 degrees per minute) and permit the hawser stresses to exceed 5 tons. For reverse heads of 2 to 3 ft the hawser stresses would be in the range of 7 to 8 tons for the faster speeds of operation. For reverse heads in excess of 3 ft, a rare condition, the lock would have to be filled by barely cracking the gate open and allowing a longer time to fill.

## Concluding Remarks

The preceding examples are indicative of the information that can be secured from hydraulic models of locks having sector-gate filling and emptying systems. It is a relatively simple matter to ascertain the effect of filling and emptying times on turbulence and on various arrangements of tows within the lock chamber. General items of interest applicable to other features of lock designs also are emphasized.

The lock studies discussed have shown that in future sector-type lock designs, care should be taken to insure that the closing force will be slightly predominant so that the gates will be tightly closed when not in use. Also loop culverts as a supplementary filling system are now recognized as an important adjunct to lock design, particularly if heads are in the range of 5 to 20 ft and the time of filling is critical. Tests to evaluate filling or emptying schedules should be conducted with a tow in the lock chamber as the time involved is a function of the depth of water cushion under the tow.

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# THE PORT OF CHICAGO[a]

Austin E. Brant, Jr.,[1] J. M. ASCE
(Proc. Paper 1768)

## SYNOPSIS

This paper presents the results of a detailed economic and engineering study of The Port of Chicago and the effects on the Port of the St. Lawrence Seaway. The methods used in conducting the study and determining the future potential general cargo commerce of the Port, the criteria developed for general cargo terminal facilities at Chicago, and the capacities of existing and proposed terminals are also discussed.

## INTRODUCTION

As a site for port facilities, Chicago's location at the southern end of Lake Michigan is centralized within the intensively developed industrial and agricultural area of the nation. Chicago is the continental hub of the nation's land and air routes and is at the junction between the two major inland waterway systems of the continent: the Great Lakes-St. Lawrence River system and the Mississippi River system. Even though ice restricts navigation on the Great Lakes and the St. Lawrence Seaway to eight months each year, Chicago is a year-round port for barge traffic to and from the Mississippi River and Gulf Coast areas.

The Port of Chicago, as defined by the Corps of Engineers, includes Chicago Harbor and Calumet Harbor on the lake front, the Chicago and Calumet Rivers, Lake Calumet, the Chicago Sanitary and Ship Canal and the Calumet-Sag Channel.

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1768 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 4, September, 1958.

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1. Project Engr., Tippetts-Abbett-McCarthy-Stratton, Cons. Engrs., New York, N. Y.

## Foreign Shipping Services at Chicago

Nineteen scheduled shipping lines currently connect Chicago with North European, Scandinavian, United Kingdom, Mediterranean and Caribbean ports. During 1956, 264 vessel sailings to overseas ports were made from the Port of Chicago. Chicago is the terminal port of most liners entering the Great Lakes, and many companies provide weekly sailings. The presently scheduled time between Chicago and Europe is competitive with the time for rail-water routings through Atlantic seaboard ports. Chicago is thus effectively linked by water transportation services with the leading industrial and merchandising centers of the world. This linkage will be strengthened by the opening of the St. Lawrence Seaway.

## Chicago's Tributary Area

The area within which the overseas commerce of the Port of Chicago would logically originate or be destined is considered to be that area within which the overall cost of freight movements via the Port of Chicago is equal to or less than the corresponding costs via other ports, based on existing rates and charges. Chicago's tributary area so defined is limited principally by the port competition offered by the ports of Toledo, Hampton Roads, New Orleans, Houston, Milwaukee, and Duluth. For individual commodities and various overseas origins or destinations, the outer limit of this tributary area varies somewhat. In general, however, Chicago's tributary area includes the states of Illinois and Missouri and parts of Michigan, Indiana, Kentucky, Tennessee, Arkansas, Colorado, Wyoming, Nebraska and Iowa. The overseas traffic generated by this area is, of course, potential on a competitive basis to all Great Lakes, Gulf Coast, and East Coast ports due to the influence of such transportation factors as time of transit, shipping schedules, port services such as banking, freight forwarding and the like, and the availability of adequate facilities.

The tributary area of the Port of Chicago covers an area of about 330,000 square miles and includes many of the agricultural, industrial and population centers of the continental United States. In this territory live over 24 million people, constituting one-seventh of the nation's population. The City of Chicago alone has a population of 3.8 million.

As a major force in the national economy, the output of goods and services of Chicago's hinterland is expected to continue growing at a pace equal to the national average, as measured by the Gross National Product. Based on studies made by Federal Government and several private agencies, it is estimated that the Gross National Product, and therefore the output of goods and services of the Chicago hinterland, will increase in the order of 35% by 1965 and 85% by 1975. This future development of the hinterland will be reflected in similar increases in the shipping activity generated by the area.

## Present and Potential Commerce at Chicago

The Port of Chicago is one of the major ports of the United States, and in 1955, handled nearly 39 million tons of waterborne commerce. In terms of total tonnage handled, the Port ranks seventh among all U. S. ports and second among all Great Lakes ports.

The major portion of the commerce at all Great Lakes ports is bulk cargo such as ore, limestone, grain, sand and gravel, and coal. At Chicago in 1955 about 94% of the total waterborne commerce was bulk cargo and about 6% was general cargo, such as newsprint, iron and steel products, cement and scrap.

## Present and Potential General Cargo Commerce

An estimated total of 2.5 million tons of waterborne general cargo commerce was handled at the Port of Chicago in 1955. Of this total, 36% or about 900,000 tons, moved to or from overseas or U. S. and Canadian lake ports in deep-draft vessels. The remaining 64%, or about 1,600,000 tons, was moved by barge.

The total overseas general cargo commerce presently generated within Chicago's tributary area includes both traffic now moving through the Port of Chicago and that having an origin or destination in the tributary area but now moving through competitive ports. To determine the volume of commerce generated within Chicago's tributary area and moving through other ports, a detailed analysis was made of confidential data on the origins and destinations of rail-borne export and import traffic. These data were obtained from the Illinois Central Railroad (New Orleans, Gulfport, and other Gulf and Florida ports), the New York Central Railroad (New York), the Pennsylvania Railroad (New York, Philadelphia, Baltimore), the Baltimore and Ohio Railroad (New York, Philadelphia, Baltimore), the Erie Railroad (New York), the Western Maryland Railway (Baltimore), the Norfolk and Western Railway (Norfolk), and the Chesapeake and Ohio Railway (Newport News). Each of these railroads furnished origin or destination information on its traffic at major ports for representative periods. The data were then expanded to represent annual volumes of commerce, and analyzed to determine the portion of the traffic now moving through the Atlantic and Gulf coast ports which is generated within Chicago's tributary area.

Combining the results of this analysis with the existing overseas traffic of the Port of Chicago, it is estimated that the annual total overseas general cargo traffic generated within Chicago's tributary area as of 1955 exceeds 1.4 million tons. This compares with an actual volume handled at the Port of Chicago in that year of 214,000 tons.

Based upon the distribution of manufacturing employment in the tributary area, it is estimated that about 70% of the total overseas general cargo commerce of 1,424,000 tons is generated within 100 miles of Chicago, 85% within 200 miles, and 95% within 300 miles. The future volume of overseas general cargo commerce generated by the tributary area should reflect its economic growth. This growth indicates a possible increase in the total overseas general cargo commerce generated within Chicago's tributary area in the order of 45% by 1965 to a total of 1.9 million tons, and 85% by 1975 to a total of 2.6 million tons.

The proportion of the general cargo commerce generated in the tributary area which may reasonably be expected to move through the Port of Chicago after the opening of the St. Lawrence Seaway is affected by the seasonal nature of seaway and lake navigation and the consequent retention of traffic by year-round ports, the possible lack of complete world-wide shipping service (at least during the early years of Seaway operation), and the reluctance of shippers to alter traditional methods of transportation and distribution. Based on

studies of the effect of these important factors, it is estimated that approximately 25% of the total overseas general cargo commerce generated within the tributary area may be considered as potential to the Port of Chicago in 1960 (immediately after the Seaway's opening) and about 50% in 1975. On this basis, the total potential overseas general cargo commerce of the Port of Chicago could reach 380,000 tons in 1960, rising to 700,000 tons in 1965, and 1,500,000 tons in 1975.

### Present Bulk Cargo Commerce

Almost 36.5 million tons of Chicago's waterborne commerce is made up of bulk commodities such as coal, iron ore, petroleum and petroleum products, sand and gravel, crushed rock, and limestone. Other bulk commodities handled in lesser volumes include grain, sulphur, sugar, molasses, salt and non-ferrous ores. Most of this volume of bulk cargo is handled at private terminals operated in connection with manufacturing and service industries such as steel mills, refineries, building material suppliers, and public utilities. These industries may be expected to expand their individual marine terminal facilities as required for their own needs.

## Existing Public General Cargo Handling Facilities

Five public terminals are now in use for handling general cargo to and from deep-draft vessels at Chicago. All are provided with railroad service and all have adequate access to nearby streets and highways.

Navy Pier is the only general cargo terminal at the Port which may be reached by ships directly from Lake Michigan without transit through locks or congested river channels. Because of its excellent vessel access and its proximity to the downtown Chicago commercial center, the site is a choice location for general cargo operations. The Pier, built by the City of Chicago in 1914, is about 3,000 feet long. The inshore 2,340-foot section was planned for use by passenger and cargo vessels, while the outer 660-foot section was built for recreational purposes. The cargo-passenger portion of the Pier is 292 feet wide and has two sheds separated by an open area about 80 feet wide which is used as a roadway and for rail and truck loading. These sheds are each two stories high, with the first floors 100 feet wide and the second floors 68 feet wide. The second floors of both sheds, originally designed for passenger use, are leased on short-time bases for exhibition purposes. The pier is not well arranged for general cargo operations since the narrow aprons (7-1/2 feet wide) and the restricted overhead clearance in the sheds (12 feet) prevent the effective use of modern cargo handling equipment. The structure is in fair condition and is suitable for improvement.

The South Water Street Terminal is located beneath the section of Lake Shore Drive parallel to the Chicago River. The storage space at the terminal is open, although part of it is protected from the weather by the elevated structure of Lake Shore Drive. The terminal site will be occupied in the future by the planned extension of Wacker Drive.

The International Steamship Terminals' facility on the Calumet River at 95th Street has handled more general cargo than any other terminal in the Port up to this time. The terminal is close to Lake Michigan so that ship transit time between the terminal and the lake is about one-half hour.

Calumet Harbor Terminals, Inc., is located at the entrance to Lake Calumet, approximately six miles inland from Lake Michigan. The average time for a one-way trip on the Calumet River between Lake Michigan and this terminal is 2-1/2 hours, and fourteen bridges must be opened to permit the passage of ships. The facilities provided at this terminal include a transit shed, warehouses, and a large open storage area.

The Chicago Regional Port District's development is located on the south shore of Lake Calumet, adjacent to Calumet Harbor Terminals. The District's facilities are among the most modern and efficient in the United States. The terminal features wide aprons provided with railroad tracks, spacious transit sheds, with single rows of interior columns, high roofs, excellent office facilities, and covered truck and rail loading docks, and a modern warehouse. All buildings are protected by automatic sprinkler systems.

## Practical Operating Capacity of a Modern General Cargo Terminal

The practical operating capacity of a general cargo terminal is determined principally by two factors:

1. The extent of the utilization by vessels of berths at the terminal.
2. The prevailing rate of loading or discharging cargo.

The navigation season of overseas vessels on the Great Lakes averages about eight months and is determined by the opening and closing of the St. Lawrence and Welland Canals and by ice conditions on the Gulf of the St. Lawrence. The operating season at Chicago is shorter than the period in which the canals are open to navigation by at least the time of a vessel's travel from and to the canals. The overseas shipping season at Chicago averages 210 days.

In the design of public terminal facilities at a port, consideration must be given to the pattern of ship arrivals and departures. Since vessel movements cannot be scheduled to insure continuous berth occupancy, a port must have sufficient terminal capacity to meet the peak demands of shipping. At Chicago, these peak demands can be expected to occur at the beginning and end of each shipping season. Studies at many United States ports, where adequate general cargo terminal capacity is available to meet peak demands indicate that, on the average, general cargo vessel berths are occupied about 50% of the time by ships being loaded or unloaded. Using a 210 day shipping season for overseas vessels at Chicago, and allowing for vessels not being worked on Sundays and holidays, general cargo berths in the Port should be designed for an average utilization of about 88 days per year. At present, berth occupancy in Chicago varies from less than 15% at some general cargo terminals to more than 50% at others.

The rate of loading and discharging depends on several variable factors including vessel size, types of cargo, number of hatches in the ship, total tonnage to be handled at each hatch, type of handling equipment used, availability and size of stevedore gangs, and the productivity of longshore labor. An average rate of 25 tons per gang per working hour is considered to be reasonable at a modern terminal. A working day for large modern vessels averages about 40 gang-hours, which may consist of 5 gangs working 8 hours each, 2 gangs working 20 hours each, or any similar combination. On any given day, of course, the number of working gang-hours per vessel may vary widely

from the average. Using the average of 40 gang-hours per vessel per working day, the daily rate of cargo transfer would be about 1,000 tons per berth. It is doubtful, however, that this vessel handling rate could be maintained at the Port of Chicago since many of the existing smaller vessels, which have only four hatches, will remain in lake-ocean service after the opening of the Seaway. The vessel handling rate would be limited also by the small cargoes moving to and from Chicago, since the handling of small cargoes entails delays due to the need to change the rigging of ship's gear for different commodities handled and the more frequent opening and closing of hatches. Allowing for such reductions in cargo transfer rates, it is estimated that a volume of 800 tons per working day for each berth would be possible for operations at modern general cargo terminals at Chicago. On this basis, the practical capacity of a single berth would be about 70,000 tons per year.

### Capacities of Existing Public General Cargo Terminals

The practical operating capacities of the existing public general cargo terminals now in use at Chicago were estimated on the basis of a theoretical berth length of 500 to 550 feet to conform to the requirements of the larger vessels expected at Chicago upon completion of the St. Lawrence Seaway. For each such berth, except the berths at Navy Pier and at the South Water Street terminal, an annual capacity of 70,000 tons was assumed. Lower capacities were estimated for Navy Pier and the Water Street terminal due to the existing physical limitations of these facilities, as discussed previously. The total practical operating capacity of the existing public general cargo terminals is estimated to be 770,000 tons annually.

In the future, the South Water Street terminal will be eliminated since the site it occupies will be required for construction of the proposed extension of Wacker Drive. At that time, the total estimated operating capacity of Chicago's public general cargo terminals should be reduced to 720,000 tons per year to reflect the loss of this facility.

### Need for Additional Public General Cargo Terminal Capacity

The existing public general cargo terminals at Chicago are now handling only a small portion of the total general cargo commerce generated in the Port's tributary area. The minimum additional public general cargo terminal capacity which must be provided at the Port to handle the potential commerce is 130,000 tons by 1965 and 960,000 tons by 1975.

## Desirable Location for General Cargo Terminals in Chicago

The principal locational requirement for a general cargo terminal serving deep-draft shipping is safe, rapid and unobstructed access for modern vessels. The sites available in Chicago for deep-draft general cargo terminals include Chicago and Calumet Harbors on the Lake Michigan shore, the Chicago and Calumet Rivers and Lake Calumet. Due to the longer time required by vessels to reach their berths and navigational difficulties, vessels sailing to and from the inland sites at Chicago (the Chicago and Calumet Rivers and Lake Calumet) must bear substantial costs which are not incurred at Lake Michigan lakefront locations.

The higher costs for vessels calling at terminals on the Calumet River, the Chicago River, and Lake Calumet reflect the delays and difficulties of navigation to these sites as compared to lakefront sites, and, together with navigational dangers of the inland passage, emphasize the advantages of the lakefront as a general cargo terminal location at the Port of Chicago. The Lake Michigan lakefront should be the site of any additional public general cargo terminal facilities required at Chicago if it is to meet competitively the challenge of other lake ports with the opening of the Seaway.

## Functional Requirements for General Cargo Terminal Facilities

Modern general cargo terminals should provide for rapid, efficient and economical transfer of cargo between ships and land carriers. Terminal design should be sufficiently flexible to accommodate evolutionary changes in methods of cargo handling, cargo transport, and terminal operations.

### Types of Berths

General cargo berths are of three general types: slip-type wharves located on slips dredged into the shore; marginal wharves paralleling the shoreline; and piers projecting out from the shoreline. At most U. S. ports, existing piers are narrow, restricting rail and truck operations on the piers. For this reason, these existing piers are usually less efficient than marginal or slip-type wharves where adequate space is provided. A modern pier which is wide enough to provide adequate space is as efficient as a marginal or slip-type wharf.

In Chicago, the Lake Michigan shoreline available for marine terminal development is limited. To provide as many berths as possible within the available length of shoreline, piers of adequate width should be used.

### Length of Berth and Depth of Water

Terminal facilities should be arranged to provide for the safe navigation and berthing of modern vessels of the types which will call at Chicago after the completion of the St. Lawrence Seaway. The Seaway locks will have a usable length of 768 feet, a width of 80 feet and a minimum depth over the sills of 30 feet, while the channels will initially be dredged to a minimum depth of 27 feet.

It is estimated that typical overseas general cargo ships serving Chicago will be of the C2-S-B1 type with a length of about 460 feet, a beam of about 63 feet, and a fully-loaded draft of about 25-3/4 feet. Some shipping interests indicate that larger vessels having overall lengths of about 480 feet will be placed in service to Chicago. For these vessels, a berth about 550 feet long would be necessary. Some of the existing lake-ocean vessels, which are about 250 feet long, will remain in service. A berth length of 300 feet would be needed for these vessels.

Since vessel drafts will be limited by the depths in the St. Lawrence Seaway channels and locks, terminal structures should be designed for an ultimate depth of water of 32 feet (including 2 feet of overdredging) below low water datum and initial dredging should be to 29 feet (including 2 feet of overdredging) below low water datum.

Width of Pier Apron

The width of the apron between the face of the pier and the transit shed should be about 55 feet. This would permit installation of two rail tracks spaced at about 20 feet to allow clearance for the swing of a locomotive crane operating on the shipside track, while permitting unimpeded movement of cargo handling equipment and trucks on the apron.

Transit Storage Space

Transit storage facilities are necessary at a general cargo terminal to hold temporarily cargo which is in transit between ships and land carriers. The amount of cargo stored at a terminal depends on the rates at which cargo is delivered or received at the terminal, and on the arrival and departure patterns of vessels, rail cars, trucks and barges. Studies of cargo movements under critical berth occupancy conditions indicate that a maximum storage capacity of about 3,800 tons of general cargo per berth would be required at a modern terminal in the Port of Chicago.

General cargo exports and imports average about 70 cubic feet per ton. Most of this cargo is palletized and stacked three tiers high to a maximum of about 13-1/2 feet. Allowing for short stacks and unpalletized cargo, the average net stacking height would be about 9-1/2 feet. To store 3,800 tons of cargo would require a net area of about 28,000 square feet. Inasmuch as space which had been cleared of cargo could not always be immediately refilled, and due to operational considerations which call for storage of cargo at locations as close as possible to the hatch to be worked, this net area should be increased by about 20 per cent, or to a total of 34,000 square feet to provide for optimum efficiency in cargo storage operations.

To work this storage space effectively, aisles for mechanical cargo handling equipment should be at least 20 feet wide. A typical arrangement for 34,000 square feet of storage area would require about 26,000 square feet for aisle space, so that the total space requirements for storage and handling of cargo would be about 60,000 square feet per berth. If highway trucks are to be operated within the shed, additional space is required.

Transit storage space should extend at least 350 feet along the vessel berth so that it would be readily accessible to every hatch on a typical C2-type vessel. For optimum efficiency in cargo transfer operations, shedded space should be at pier deck level in single-story structures. Within the sheds the widest possible column spacing consistent with economy of construction should be used. Overhead clearance within sheds should be about 20 feet for unimpeded stacking of cargoes. The shed floor and wharf apron should be designed to support a load of 600 pounds per square foot.

Railroad and Truck Loading Accommodations

Based on an annual cargo handling capacity of 70,000 tons per berth, an average of 400 tons of cargo per day would be moved to and from each berth by railroad cars and trucks. On the basis of past experience at Chicago terminals handling overseas cargo movements, it is estimated that about one-half the general cargo would be moved to and from the terminal by rail, while the remaining one-half would be handled by truck. This is equivalent to an average daily movement of 200 tons by rail and 200 tons by trucks. During peak periods, especially at the beginning and end of each year's shipping

season, the combined rail and truck movements to and from a terminal might be as much as 100% greater than the average daily movement or about 800 tons per day for a single berth. While, on the average, the rail and truck movements would be about one-half this total, it is estimated that, on any given day, either rail or truck movements may be as high as 480 tons per berth.

Rail Loading.—For an average general cargo carload of about 20 tons, the peak daily movement by rail of 480 tons per berth would be equivalent to about 24 carloads. With two rail switching movements per day (such as is customary at general cargo terminals), space would be required for 12 rail cars at each berth.

Truck Loading.—For an average general cargo truck load of about 7 tons, the peak daily movement by truck of 480 tons per berth would be equivalent to 69 truck loads. With an average truck loading time of about 2-1/2 hours and a working day of eight hours, a maximum of three trucks could be handled per day at each truck loading position, and space would be required for at least 23 trucks at each berth. During parts of a peak day, such as at the start of work in the morning and immediately after the lunch period, more than 23 trucks might desire to load or unload cargo.

## Recommended Terminal Development Plan

Preliminary cost estimates for terminal construction at Chicago's two lakefront harbors—Calumet Harbor and Chicago Harbor—indicate that the most economical initial development to meet current requirements for public general cargo terminals can be met by the improvement of the existing Navy Pier and subsequently by the construction of a new pier south of the Chicago River controlling works. Construction of terminal facilities in Calumet Harbor may, however, be required in the future when the demand for general cargo berths in Chicago exceeds the number that can be provided economically in Chicago Harbor.

Under the recommended plan, the south side of Navy Pier would be improved to provide a modern four-berth general cargo terminal having a combined operating capacity of about 280,000 tons a year. Each berth would be 550 feet long. In the initial development at Navy Pier the south side of the pier would be improved to provide two modern general cargo berths.

The initial improvement of Navy Pier would consist of three phases: widening of the pier, rehabilitation of the existing shed and construction of a new shed. The widening of the pier would be accomplished by constructing a 96-foot wide pile-supported concrete deck 1100 feet long on the inshore end of the pier. The wharf structure would be designed for dredging in the adjacent berth to 30 feet below m.l.w.

Since the existing first floor in the south shed of Navy Pier is structurally inadequate to support the 600 lb. per square foot loading required for transit shed operation, extensive rehabilitation of column footings and floor supports would be required. A combined rail and truck loading platform would be provided at the rear of the existing shed. Two rail tracks would be provided at this platform. The platform would be adequate to handle 23 trucks and 12 rail cars simultaneously at each berth. Two rail tracks would be provided also on the apron.

For each berth the existing shed would have an area on the first floor of about 48,000 square feet per berth. Since the existing shed area is restricted by columns spaced at 16-1/2 feet transversely and 20 feet longitudinally and by an overhead clearance of 12 to 13 feet, it is estimated that the effective area for transit shed operation of that shed would be only about 32,000 square feet per berth, about 1/3 less than the actual area. Since about 60,000 square feet per berth would be required for effective shed operation at Chicago, a new shed would be built to provide 28,000 square feet of space per berth on the widened portion. The new shed would be 48 feet wide and would extend the full length of the two berths immediately behind the 55 ft. apron.

The construction cost for the initial improvement of two general cargo berths on the south side of Navy Pier is estimated at about $4,000,000. In addition, deepening of the existing federally maintained Chicago Harbor entrance channel and construction of a connecting channel to the south side of Navy Pier would cost about $1,500,000. The two improved berths at Navy Pier would have a combined annual operating capacity of 140,000 tons of general cargo per year.

The second stage of development in Chicago Harbor would be the improvement of two additional general cargo berths on the south side of Navy Pier. The improvement would be similar to that for the initial two berths and would cost about $4,000,000.

The third stage of development in Chicago Harbor would be a modern nine berth general cargo terminal, having a combined annual capacity of about 630,000 tons, which would be constructed on a filled pier 2,400 feet long and 650 feet wide immediately south of the Chicago River. Vessel access would be provided by dredging slips on both sides and at the end of the pier, and connecting them to the existing Federally-maintained Chicago Harbor channel. Vessels berthed on the north side of the pier would be protected from wave action by the existing Chicago Harbor breakwaters, the Chicago River lock walls, and the pier itself, but vessels berthed on the south side and on the end would be exposed to waves coming from the southeasterly direction. To provide protection for these vessels, it would be necessary to construct a new outer harbor breakwater.

Four single story transit sheds, each serving one berth and having 60,000 square feet of transit space, would be built on each side of the pier immediately behind a 55-foot wide apron. Each berth would have loading platforms adequate to handle 26 trucks and 16 rail cars simultaneously. With a wide pier of this type truck and rail access would be arranged so that there would be no interference. The total construction cost for the development of nine general cargo berths south of the Chicago River would be about $15,500,000. In addition, the cost of the required harbor improvements would be about $12,500,000.

The 13-berth construction program recommended for general cargo facilities in Chicago would provide sufficient terminal capacity until about 1973. After that time, it may be necessary to construct additional terminal facilities to meet the growing needs of waterborne commerce.

## ACKNOWLEDGEMENT

The data for this paper were obtained from a detailed economic and engineering study of the Port of Chicago conducted in 1957 by Tippetts-Abbett-McCarthy-Stratton. The study and the detailed designs and construction plans

now being prepared for the initial two-berth development at Navy Pier are under the direction of the Department of Public Works, which is responsible for port operations within the City of Chicago.

Acknowledgement is due to the many individuals who furnished valuable information. The cooperation of the Illinois Central Railroad and the Chicago and North Western Railway in furnishing traffic data and information on rail access to the proposed developments was invaluable. The assistance of the Corps of Engineers, especially the Great Lakes Division Engineer and the Chicago District Engineer and their staffs, is gratefully acknowledged. The officials of many departments of the City of Chicago, Cook County, and the State of Illinois and the officials of the Metropolitan Sanitary District of Greater Chicago, the Chicago Regional Port District, and the Chicago Park District were most helpful. Special acknowledgement is made also of the cooperation of the officials of the City of Chicago Department of Public Works and the Bureau of Rivers and Harbors. This paper owes much to the whole-hearted cooperation of these and many other organizations and individuals.

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# CHANGING SITE REQUIREMENTS FOR PORT OPERATIONS[a]

Peter Engelmann,[1] A.M. ASCE
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## INTRODUCTION

Site selection and site adaptation for construction of port terminals require the evaluation of natural and man-made environment for each specific project. Common to most projects, however, is the need for analyzing space requirements of anticipated future operations, to ensure adequacy and economy in the transfer of cargo during the life of the installation.

Recent developments in design of ocean vessels and cargo-handling methods have brought about changes not only in navigational requirements for channels and harbor basins, but also in shore facilities. There is no indication that these developments are at a stand-still. In this paper, an attempt is made to define some aspects of land area requirements for future port operations.

### Changes in Criteria and Approach

Site planning involves the evaluation of real estate in the form of existing land, or land to be created, whose long-term value will depend solely on its utility. Where substantial capital investment is required for the acquisition or creation of such land, this investment must be justified; similarly, where a site presents itself which provides deep-water access to inland transportation without adequate real estate, the disadvantages of constructing a terminal there must be explained.

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a. Presented at the Sixth International Conference on Coastal Engineering of The Council on Wave Research, The Engineering Foundation, December 2 - 7, 1957 at the University of Florida Gainesville, Florida.

1. Project Engr., - Parsons, Brinckerhoff, Hall and Macdonald Engrs. - New York.

Unfortunately, the examples for capacity utilization of large land areas at existing terminals are few, the only cases with long performance records being bulk cargo terminals, for the most part highly specialized and mechanized. Among general cargo terminals with which this paper is primarily concerned, those now having large areas tend to give a distorted picture about optimum land utilization, either because of bottlenecks inherent in their present mode of operation or because of insufficient volume in cargo movement. This lack is aggravated by the many existing general cargo terminals with small land areas, which give the impression of being adequate since, it is argued, somebody handles cargo there at a profit.

However, the frequency of costly congestion and delay at piers with limited operating areas has already caused a general change in approach toward problems of site selection and site layout for new terminals. The mere number of vessel berths is no longer regarded as a true measure of the cargo-handling capacity of a port and greater emphasis is now given to the land available to "support" each berth. The higher rate of cargo movement through the major terminals of today, occasioned by the use of larger vessels and palletized handling methods, has resulted in a larger scale of landbased transfer operations; presently, the cost of these operations on land may amount to as much as one half of the total cost of cargo transportation between ports.

Despite this change in emphasis, the vessel berth itself remains the starting point of all considerations in site planning. Its location relative to existing deep water and to land transportation lines determines the configuration of the entire facility, and the high initial cost of wharf and pier construction dictates provision of adequate real estate, to allow for ultimate maximum capacity of cargo transfer at each berth.

This ultimate maximum capacity will vary, of course, with the type of traffic at every locality. 150,000 to 200,000 short tons of mixed general cargo per year has frequentiy been taken as a design figure for berths in the United States; this applies where the type of traffic allows relatively high utilization of berthing space. Vessels moored and working at a berth fifty to sixty per cent of the operating days in a year is generally considered maximum utilization for the average port, unless the traffic involves a regularly scheduled, shortrun service. While many ports cannot now approach such high berth utilization, it is in the nature of the ocean shipping business that, when heavier traffic does develop, existing berths will be subjected to such use, regardless of the adequacy or inadequacy of supporting shore facilities. The sporadic nature of waterborne traffic will, of course, result in much higher than yearly average utilization of berthing space in recurring periods of several weeks or months.

Even assuming that the present average berth utilization by vessels remains unchanged, which is probably justified, design criteria for future shore facilities will change drastically. Present cargo operations at the waterfront are hampered by shortcomings in handling equipment, packaging of cargo, and labor management practices; when these shortcomings are overcome, as is likely to happen within ten or fifteen years, this may allow vessel turn-around in half the time, so that 3000,000 to 400,000 short tons of general cargo per year could then be handled at a single berth. Consequently, for future movement of a given tonnage through the port, the required number of berths will decrease and the amount of land area needed to adequately support each berth will increase.

## Uses of Land Areas

At general cargo terminals, the land must accommodate the following categories of operations:

(1) Transit handling and transit storage of cargo
(2) Access for means of land transportation
(3) Long-term storage of cargo
(4) Auxiliary services, such as administration, parking fields, maintenance shops and personnel facilities

Of these categories, requirements for long-term storage and auxiliary services will vary widely for each locality. With regard to access, it hardly needs emphasis that land traffic will increase proportionately with cargo traffic at the berths, but it should be noted that there is no forseeable permanency in any specific distribution of traffic between rail and truck, where both are available at a terminal. Present trends in most United States and many Latin American ports are toward increased truck movement to and from the waterfront. Nevertheless, wherever possible, allowance should be made for access space in which the majority of all estimated future cargo traffic either by rail or by truck could be handled.

The transit areas are, perhaps, the most critical category among the land allocations within a port, in view of both their initial cost and their potential future effect on terminal efficiency. For purposes of this discussion, the transit area of a berth is defined as any land or pile-supported deck serving cargo operations which can directly influence the rate of ship loading or ship discharge; this includes not only short-term storage, sorting and inspection areas for cargo, but also the truck and railroad operations serving them in the vicinity of the berth, and such auxiliary services as may be associated therewith.

## Size of Transit Area

In determining the size of transit areas, a distinction should be made between the requirements for transit storage of cargo and those for transit handling.

Transit storage area, the square feet of dock actually occupied by cargo while at rest during this part of the handling cycle, represents a type of reservoir necessary at every transfer point between intermittent and continuous flow of materials. In this instance, the ocean vessel constitutes the intermittent flow phase and land transportation represents continuous, though not uniform flow. Transit storage is necessary, but it is not, to any appreciable extent, in itself revenue producing.

The transit handling portion of the total transit area consists of all land or deck not used for transit storage or associated auxiliary services. The aisles, aprons, loading platforms, truck spots, roadways and railroad tracks "support", in the physical as well as in the operational sense of the word, the movement of cargo; these areas should, therefore, be considered as revenue producing.

Transit storage requirements can be calculated, with some degree of accuracy, if it is known how much cargo will be at the terminal at any one time. Unfortunately, the transit handling areas cannot be derived in the same manner; the configuration of ther terminal, the type and arrival pattern of land

traffic, and the handling methods used all have a bearing on space requirements and, for the final design, should be analyzed and developed in detailed layouts, before the areas can be determined. However, in most instances, neither the time nor the essential basic data on cargo traffic are available during the site planning stage, for definition of these operational features. It therefore becomes necessary to allocate acreage on the basis of experience gained at existing facilities, and by general forecasts of future needs. The following three examples may be of interest.

(1) Port of New York Authority Designs - Criteria for covered transit area at general cargo berths have been developed by the Port of New York Authority (Ref. 1), for terminals where 12,500 measurement tons (or approximately 8,000 short tons for average cargo at that port) must be accommodated per berth. For transit storage, these criteria indicate 45,000 sq. ft.; for transit handling, including aisles and loading spots for thirty trucks, another 45,000 sq. ft. are added, so that the total covered transit area, exclusive, however, of aprons, truck waiting lines and approaches, amounts to 90,000 sq. ft. per berth. In the explanation of these criteria, necessary reservations are made as to the limitations in applicability for other ports, and it is noteworthy that, for the most recent addition to the Port Newark Terminal, the Port of New York Authority is providing 88,000 sq. ft. of covered transit area, exclusive of open storage and provisions for truck loading along platforms entirely outside the shed; the total transit area at this terminal will be approximately six acres per berth.

(2) Private Steamship Terminal Study - Storage area requirements appear to be very much affected by the customs and practices of each individual terminal operator. An evaluation of such specific space needs, based on analysis of cargo records for a two year period, has been made in a recent study of one major steamship terminal at a north atlantic port, where two berths were being utilized nearly to capacity by C-2 and Mariner type vessels. Calculations indicate that 175,000 sq. ft. of storage area alone would be required to accommodate the cargo on dock during the average of peak traffic conditions occurring 25% of the year. The total required transit area, based on preliminary design layouts for this particular facility, which also had to provide for extremely severe peaks of truck traffic, amounted to nearly twelve acres. This area would allow for capacity utilization of the two berths, with vessel-loading and discharge by the customary methods of today, at an average annual rate of between 150,000 and 175,000 short tons per berth.

(3) Data Compiled by A.A.P.A. - An interesting tabulation of total operating areas at several existing, modern terminals on the East and West Coast, has recently been prepared by the American Association of Port Authorities (Ref. 2). The areas listed vary from little over two acres to more than nine acres per berth, and the additional data are indicative of the wide range of utilization that is experienced in current operations at these terminals, the annual cargo movement varying from 56.5 to over 300 tons per linear foot of berth.

The foregoing examples deal with the normal general cargo facility, where the majority of imports and exports are handled in cases, crates, bales or bags of varying sizes and shapes. However, some of the fully containerized operations that are presently in the development stage, and the projected roll-on, roll-off methods, while serving traffic of the same general cargo,

will have entirely different requirements for transit areas at the terminals.

A container type operation under study by one U. S. shipping company, will require a minimum depth of 480 feet behind each berth, or approximately ten acres, for transit storage and handling; sorting and consolidation of cargo for this container operation would take place at upland areas not included in this acreage. It is estimated that such a berth could handle in the order of 1,500,000 short tons per year.

According to preliminary studies by the Maritime Administration (Ref. 3), transit and marshalling areas for future train-ship for trailer-ship terminals will be even greater. Space requirements for vehicles in transit will depend on the sizes and sailing schedules of vessels; allowances of 1,000 sq. ft. per trailer and 900 sq. ft. per railroad car stored in transit areas are recommended by the Maritime Administration for general land allocation purposes, and mention is made of total areas in excess of thirteen acres per berth.

Evidently, accurate forecasts of space requirements that would apply to any and all future general cargo berths, cannot be made; but it is certain that the needs will be much greater than the land areas that have been provided at most general cargo terminals in the past. For new construction, four acres per berth appear to be indicated as a minimum for transit storage and transit handling alone, while six to eight acres per berth can probably be utilized to capacity at most major ports within the very near future. Where general land allocations also include long-term storage and auxiliary port services, twelve to fourteen acres per berth would not be at all excessive.

## Location and Cost of Transit Areas

Frequently, the cost of providing necessary acreage for cargo operations is very much dependent upon the location of transit areas in relation to water areas. At conventional general cargo terminals, it is most desirable that the entire transit area be directly adjacent to the berths, whereas at bulk cargo terminals, transit storage can often be located several hundred feet or, in the case of liquid cargos, several thousand feet inshore. For roll-on, roll-off operations, the distance between trailer parking fields and berths is not as critical as for conventional general cargo. Trailer operations, even for side loading or lift-on, lift-off vessels, can be economical at narrow, open finger piers. For container operations, it appears to be only a matter of solving some relatively simple problems of mechanization, until this type of cargo can also be handled at berths of the finger pier type, with large transit areas located a considerable distance further inland.

Regardless of location, the cost of providing the necessary land area for future cargo operations often presents difficulties. The initial capital investment for pile-supported deck construction, bulkheading, and other items of limited life must be amortized within a reasonable period, while interest must be paid on the cost of permanent assets created, such as those of the land itself. In many ports, the resulting annual charges have to be covered by income from terminal rentals,and the first tenants may be unwilling to pay for the cost of land they cannot fully utilize in the immediate future. Where the method of financing and the administrative structure of the port allow payment of these charges on the basis of tonnage actually handled at the terminal, these problems seem to be less severe.

In conclusion, it should be mentioned that it is very much within the scope of work for engineers involved in site planning, to insure that neither the financial or administrative structure of the port, nor erroneous comparisons with older terminals presently operating on an acre or two per berth, prevent construction of new terminals with the much larger land areas that will be necessary for future port operations.

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# THE SUEZ CANAL—ITS CHRONICLE AND BIBLIOGRAPHY

Shu-T'ien Li,[1] M. ASCE
(Proc. Paper 1770)

## SYNOPSIS

This paper presents the chronicle of important development and engineering events of the Suez Canal beginning from nearly 4,000 years ago to the Egyptian nationalization of 26 July 1956, and a chronologically arranged bibliography of published works and records thereon from about 1800 to the end of 1957. The historical account of events treated has been prudently authenticated and carried up to the proclamation of nationalization, leaving social, economic, and international aspects and what has occurred since nationalization to a later paper. To facilitate waterways engineers and those who are interested in this strategically located interoceanic sea-level canal to refer to the original sources for complete, authentic, factual information, a complete chronological bibliography has been carefully compiled.

## INTRODUCTION

Perhaps there has been no other single civil engineering project in human history that has exerted so much dynamic influence on the lifeline and economy of Western Europe, nor has any other waterway been so important in mercantile ocean commerce, nor has any other engineering feat caused as many perplexing international political, diplomatic, and strategic issues as the Suez Canal, which runs through the Isthmus of Suez in Eastern Egypt and stretches about 100 miles long from Port Said on the Mediterranean to Suez on the Red Sea. It was dug through the desert waste, swamps, and lakes of the Isthmus joining Africa to Asia. Only in a short section south of Little Bitter Lake, it followed the ancient route. Few engineering difficulties were encountered in its construction. Of the many proposed routes for interoceanic canals, it is

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1. Formerly Bd. Member, Grand Canal Impt. Planning Bd., Republic of China.

the only open sea-level canal that has been completed without locks.

The importance of the Suez Canal lies in the fact that it shortens the sailing distance between the East and the West by avoiding the round of Africa, which varies according to the journey and, in the case from Marseilles, France to Bombay, India, amounts to about 5,800 nautical miles. It, therefore, (1) increases the ship's "rotation," (2) permits a more rapid redemption of the capital outlay on the ship, (3) lessens the expenses of crew, fuel, and insurance for each voyage, and (4) spreads out over more volume of freight the costs of repair and upkeep. For a round trip from Port de Bouc to Kuweit and back, a 32-33,000-ton dead-weight ship, with an average speed of 15-3/4 knots and with a daily fuel consumption of 80 tons, would economize 2,800 tons (56 per cent) of fuel by using the Canal. Thus, it constitutes the life line of Western European maritime countries, particularly Britain and France.

In the whole human history of navigation between the East and the West, there has been no other single contribution more significant than the introduction of the mariner's compass to Europe from China by the Arabs as far back as 1,000 years ago and the excavation of the present Suez Canal by the French engineer, Ferdinand de Lesseps, shortly after the mid-nineteenth century. The introduction of the mariner's compass was through the navigators of the Arabs, but not their invention; the excavation of the Suez Canal was through the isthmus of Arabs, but not engineered by them. The magnetic compass has afforded infallible orientation with the north-and-south direction, day or night, clear or cloudy; while the Suez Canal has made "land divided and oceans unite."

This most celebrated and most used waterway of the world and the events associated therewith have attracted world-wide attention. It appears that this Society should have on its permanent technical record an account of the role this canal has manifested in the history of engineering works.

## From Prehistoric Strait to the Present Isthmus

A study of the geological formation of the Isthmus of Suez shows that all the low lying land was originated from the tertiary period and often from the quaternary period of the cenozoic era. Based on radioactive disintegration, the recent epoch of the quaternary has a duration of about 25,000 years, preceded by its pleistocene epoch of about 2 million years. Thus, the end of the tertiary period may be estimated at about 2,025,000 years ago. To the geologists, it seems to be established that at the end of the tertiary period, the Red Sea joined the Mediterranean. Hence, there existed the prehistoric Strait of Suez.

The Isthmus gradually assumed its appearance through the accumulation of deposits left there by the sea, the Nile, and the winds, and also because of slight upheavals that have taken place from time to time, for the Isthmus region cannot be considered definitely stable from a seismic point of view. Further, there is no doubt whatever that the Bitter Lakes remained in direct communication with the Red Sea even after the appearance of the Isthmus, but the period of their final parting could not be established. After the final parting of the Bitter Lakes with the Red Sea, there has not been much change in the appearance of the southern part of the Isthmus.

Violent earthquakes sometimes causing a subsidence of the land have been recorded in ancient times and in the Middle Ages, and some writers attribute

to these the formation of Lake Menzala situated for its major part on the west side of the northern end of the present canal. When the Nile had seven branches instead of the two it retains today, the lowlands lying to the north of the Isthmus formed a wide plain which received the last of the great river's fertile deposits. The Tanitic branch of the Nile had its outlet some nine miles east of where Port Said stands today, and the Pelusiac branch (the most easterly of the ancient seven branches) entered the Mediterranean still further east at Pelusiac Mouth.

The narrowest part of the present Isthmus hardly exceeds 75 miles when measured from Suez on the Red Sea to Pelusium on the Mediterranean, and the highest ridge about halfway in between at El Gisr is only some 52 to 56 feet above sea level. From El Gisr northward, start the shallow depressions, 3 feet or so below sea level, and from there southward are situated Lake Timsah, Great Bitter Lake, and Little Bitter Lake.

Despite the barrenness of the soil, the Isthmus had already asserted, ever since its appearance, the real function of uniting the African and Asiatic worlds. It acquired still greater importance during the Persian occupation of Egypt.

## Ancient Pre-Christian Era Developments From Circa 1920 B.C.

A waterway between the Mediterranean and the Red Sea was already constructed at the time of the Pharaohs of Egypt circa 1920 B.C. The early Egyptians built this first primitive version of the Suez Canal as a dual-purpose navigation and irrigation canal connecting the east branch of the Nile with the Red Sea. This Canal of the Ancients was along Wadi Tumilat which became the Biblical Land of Goshen about 1707 B.C., to Lake Timsah, thence through Bitter Lakes, and then to Suez. It served them much the same as the present Suez Canal does to the world today. The connection with the Nile, a heavily silt-bearing river, together with the very flat slope between the Nile and the Red Sea, had probably caused recurring ruins of the ancient Egyptian versions of the Suez Canal.

From about 1400 B.C. to the middle of the 13th century B.C., Egyptian kings Sethos I and Ramses II (about 1324-1258 B.C.) constructed a canal between the Nile and the Red Sea for the fleet through Lake Timsah. During the Persian occupation, the road to Persia was by way of the Wadi Tumilat following the wide navigable canal started by Rameses II for the fleet and to irrigate the fertile valley between the Nile and Lake Timsah. This canal again became unusable, apparently because of silting up as a result of a weak current from the Nile toward the Red Sea. The gradient of the Nile from Cairo to the sea drops only 3.9 in (10 cm). The east arm known as Pelusiac Branch, which was of importance in ancient times, was only about 3.28 ft (1 m) over mean sea level at a point northwest of Lake Timsah. The current was doubtless weak toward the Red Sea, and consequently the canal went to ruin in the course of time.

The first Suez Canal on the present route was of limited depth, and it was completed about 1,000 B.C. But the navigable artery which was reconstructed and extended by Necho, son of Psammitichus, about 600 B.C. was again through Wadi Tumilat. It made inland waterway traffic possible to beyond At-Tuku. After the death of Necho, some 120,000 workers, according to Darius, perished in the digging. Darius, son of Hystapsis, continued the work which had been

interrupted for some twenty years, and summoned the notables of the land of Sheba to the important city of At-Tuku on its banks. Darius left it unfinished by about 500 B.C.

As the Greek historian Herodotus (5th century B.C.), known as the Father of History, tells us, the canal joined the Pelusiac branch a little below Bubastis. First of all it followed an easterly course along the Wadi Tumilat then, beyond Heroōpolis (At-Tuku of the ancient Egyptians), turned southwards to cross the Bitter Lakes 'whose once brackish waters had become fresh through connection with the river.' The canal finally reached the Red Sea, near the site of what is now the modern town of Suez, at a point the ancients called Clysma. This waterway was navigable over the whole of its length during the flood season but it seemed to have been more important as a means of irrigation than as a trade route and never to have seriously competed with caravan routes from Berenice and Myos Hornos, which joined the River at Coptos.

Classical scholars ascribe to the Ptolemies (Ptolemy I, 367?-285 B. C., king of Egypt 323-285 B.C., first of the Greco-Egyptian dynasty, called Ptolemy Soler; Ptolemy II, 309-246 B.C., son of Ptolemy I, king of Egypt 285-246 B.C., called Ptolemy Philadelphus), who, despite their Greek ancestry, were true Egyptian sovereigns, both the completion of the Canal of Darius, which then filled the Bitter Lakes with salt water from the Red Sea, and the foundation of Arsinoē, a center of communications in the southern part of the Isthmus.

Among these works, it was, however, the canal from the Nile to the Red Sea which made the greatest impression on classical geographers. Writing circa 60 B.C., Diodorus of Sicily sums up their views in these terms: "It (the Canal) was undertaken by Necho, son of Psammitichus, who initiated the work; then it was carried on by Darius, King of Persia, who made some headway, but who left it unfinished after he had been warned that if the Isthmus were cut all Egypt would be flooded because the level of the Red Sea was higher than the level of Egypt. Finally, Ptolemy, the second of that name, put the last hand to the task and conceived the idea of having a lock built in the canal to hold the waters back; this lock could be opened at will when it was desired to sail beyond, then it was closed, and use has shown that the construction was justified."

The actual site of this canal is well known. It was discovered by Napoleon I and described by Linant de Bellefonds before modern works had altered the appearance of the area.

Thus, during the pre-Christian era from about 1920 B.C., it had already been possible for the ships of the ancient times to pass from the Mediterranean to the Red Sea by making use of the Nile up into the heart of the Delta, and thence by the 'Canal of the Ancients,' which came out at the head of the Gulf of Suez. This canal, of which certain vestiges are still visible at the present time, was often ruined, cleared and restored.

## Early Developments During the Christian Era up to 1832 A.D.

The Romans, to promote their trade, lavished even greater care on harbors, roads, and canals, and, in order to ensure a greater flow of water through the Wadi Tumilat, carried its inlet back to the vicinity of Egyptian Babylon. This new canal was apparently constructed between 98 and 117 A.D. when Trajan

(52 or 53-117 A.D.) was Roman emperor. The junction-canal came to be known as the River of Trajan. Then, by neglect, the Byzantines, who succeeded the Romans, allowed the canal to silt up and to become unusable.

Attempts of the ancient Phoenicians to rebuild the Canal was not successful.

General Amru brought it back into existence. As Governor of Egypt, during his first term of office (641-644 A.D.), he conceived the idea of a direct canal fed by the waters of the Nile and cut across the flat plain slightly above the level of the two seas, which extends east of Tell Farama, a town situated close to ancient Pelusium. But after Caliph Omar had opposed this scheme. he reopened the canal of the Ancients from Cairo to the Red Sea in six months, so that vessels could sail to Hejaz, the Yemen, and India. For more than 130 years, the canal was again in use, but near the end of the eighth century, when Mohammed Ben Abdallah Ben Hassan revolted at Medina against the Abbasid Caliph Aben Jafar Al Mansour, the latter had it filled in, in order to prevent it from being used to bring supplies to the town that had rebelled against his authority. It thus became disused in 775 A.D.

From the time the Canal of the Prince of the Faithful was filled in, the Isthmus of Suez, no longer inhabited in the north, ceased to be of any great importance. Trade between Europe and India flowed along various routes.

In the years that followed Vasco da Gama's rounding of the Cape of Good Hope in 1498 and the organization of the new route to India, it occurred to enlightened minds of the Venetians, who until then had had a large share of the Mediterranean trade, that the only rational solution would be to cut through the Isthmus of Suez and this was proposed to them as early as the beginning of the sixteenth century. In 1586, El-Enldj-Ali, Captain Pasha (or Pacha) of Turkey, which held all the lands now known as the Middle East, put forward the same idea, but in vain.

In the seventeenth and eighteenth centuries, many prominent persons gave thought to the cutting of a canal between the two seas to avoid costly overland trans-shipments. Such were the names of Colbert, of the Marquis de Nointel (French Ambassador to the Porte), of Leibnitz, of the Marquis d'Argenson, and of Baron de Tott. However, Constantinople looked with disfavor upon European schemes tending to end the monopoly of navigation in the Red Sea which was then the preserve of Ottoman subjects. Moreover, the various proposals put forward at the time did not rest on any serious technical survey of the conditions in which a navigable route between the Mediterranean and the Red Sea could be opened.

It was thus left to the French engineers who accompanied Napoleon I (1769-1821) to Egypt in 1798 to embark on the first of these surveys. In fact, one of the objects set by the Directory to the Expedition was to cut the Isthmus of Suez.

Napoleon himself travelled to Suez at the end of 1798. As he left, he recognized the remains of the Canal of the Ancients extending over a distance of some twelve miles. On his return to Cairo, he directed the Chief Engineer of the Highways Department, Jacques-Marie Le Père, to make a detailed survey of the Isthmus, and this topographical work, after several interruptions, was finally completed at the end of 1799.

As contained in the "Description de l'Egypte", the masterly work in which the result of the labours of the scholars and scientists who accompanied Napoleon I are recorded, the findings of Le Père led to the mistaken conclusion that the level of the Red Sea was some 33 feet above that of the

Mediterranean. Until the middle of the nineteenth century, these findings were, however, generally accepted as correct, constituting for some fifty years a serious obstacle to the realization of schemes for cutting through the Isthmus.

As a measure to avoid the risk of flooding part of Egypt, engineers of this period were apt to consider only a canal somewhat similar to the Canal of the Ancients, joining the Red Sea to the Nile, instead of a direct communication between the Mediterranean and the Red Sea. The canal section suggested by Le Père would have only a width of 13 m (42 ft 6 in) at the bottom and a depth not exceeding 4 to 5 m (13 to 16 ft), so that traffic would have had to cease during the season of low water in the Nile.

## Saint Simon's "Regeneration of the World," and De Lesseps' Promotion of 1832 to 1866

Viscount Ferdinand Marie de Lesseps originally seized the inspiration to construct the canal from his political and philosophical mentor, Count de Saint Simon (1760-1825), and from Saint Simon's disciple and editor, Enfantin, himself a brilliant engineer. Saint Simon thought of canals at Suez and Panama as primary factors in the "regeneration of the world." So did de Lesseps. Indeed, they are.

De Lesseps was a French diplomat and later founder of the Suez Canal, who, at the age of 27, headed with the idea for the Suez Canal Project in 1832, while he was sent to Alexandria, as vice consul, at a time when a pestilence was raging over Egypt. De Lesseps was quarantined. While confined in a repulsive lazaretto, he looked up Le Père's report to Napoleon I on the project. He proceeded to persuade his friend, Mohammed Said Pasha, the Viceroy of Egypt (1822-1863), that it was feasible.

Towards the end of 1833, the Saint Simonians landed in Egypt where they knew they would be welcomed by a monarch whom they described as the 'great promoter of the peaceful glory of battles against nature'. The Saint Simonians, who wanted to join the two seas by a canal, were permitted to lay their schemes before the government. At the beginning of 1834, the Grand Council of Egypt decided that the Delta locks should be constructed before the question of cutting through the Isthmus was considered.

Linant de Bellefonds who studied most carefully, before Ferdinand de Lesseps, the question of cutting an artificial waterway between the Mediterranean and the Red Sea, was later on in a position to place at de Lesseps' disposal a valuable file of documents which were to be extremely useful to him.

There still existed, however, a professional contention among French scholars to be disproved, namely, a then regarded difference of 29 ft between the elevation of the Mediterranean Sea and that of the Red Sea. This error was first corrected in 1841 by English officers.

In 1846, there was set up in Paris an organization called Société d'Études pour le Canal de Suez, comprising three groups: a French group with Enfantin and the Talabot brothers (Paulin Talabot and his brother), a British group with Robert Stephenson as its engineer, and an Austrian-German group with Negrelli de Moldelbe as the leading light. The Société dispatched technicians to the Isthmus to undertake a survey from the Mediterranean to the Red Sea. In 1847, with the help of Linant de Bellefonds, Bourdalone succeeded in establishing that the difference in level between the two seas was practically

negligible. This is sometimes referred to as the French scientific report which dispelled the chimera of the difference in level between the two seas. But, besides this conclusion, nothing more resulted from the labours of the Société.

Having heard of the accession of His Highness Mohammed Said, who had been his great friend in youth, Ferdinand de Lesseps decided to go to Egypt without delay and landed in Alexandria two months later on November 7, 1854. The fruit was taking shape in only 23 days later.

On November 30, 1854, Ferdinand de Lesseps obtained the first Act of Concession. He then proceeded to formulate technical plans.

On January 5, 1856, a second Act of Concession, confirming and completing that of 1854, was signed by the Viceroy, conferring upon the Compagnie Universelle du Canal Maritime de Suez the right to cut, maintain, and operate a maritime canal for 99 years from completion of the work. On expiry of this Concession, the Egyptian Government was to take possession of the Canal without paying compensation but was to purchase plant and supplies at valuation. It was subsequently laid down in additional agreements that the Egyptian Government should also purchase the dwellings used to house employees.

Thus, to the genius and singleness of purpose of Ferdinand de Lesseps, who enjoyed the support and patronage of far-sighted Egyptian sovereigns, belongs the eternal distinction of translating into factual terms that dream of nearly 4000 years, the creation of a modern direct navigable route between the East and the West.

Having obtained the concession charters, or "firmans", from Mohammed Said, the Viceroy of Egypt, de Lesseps strove to fulfill the numerous conditions that would allow work to begin.

The System of Freedom of navigation through the Suez Canal was first placed by the Firman of His Imperial Majesty the Sultan on the 22nd day of February 1866 ratifying the concessions granted by His Highness the Khédive. It was later confirmed by the International Convention of 1888 which completed the system.

## The Development of the Canal Plan, 1846-1859

From 1846 to 1856, the Austrian Engineer, Negrelli de Moldelbe, made accurate studies. After de Lesseps obtained the first Act of Concession in November 1854, he further proceeded with the profound technical studies that were indispensable.

Two French engineers in the service of the Viceroy, Linant de Bellefonds and Mougel Bey, drew a scheme for cutting a direct maritime canal across the Isthmus. This plan was for a canal 146 kilometers (91 miles) long, with a lock at each end, joining Suez to Pelusium and passing through the Bitter Lakes. The locks were objectionable.

In order to have all advisable technical safeguards, de Lesseps suggested to the Viceroy, Mohammed Said Pasha, that the advice of eminent engineers should be sought, and this led to the creation of the International Technical Commission in 1855, composed of 13 members from the 7 countries of England, Austria, Spain, France, the Kingdom of Sardinia, the Netherlands, and Prussia, whose task was to examine the Linant-Mougel plan and to draw up a definite preliminary plan. The Commission held its first meeting in Paris on 30 October 1855, when five of its members were delegated to study the question on the spot.

The Isthmus of Suez was then visited by the delegates of the International Commission which was briefed by H. H. Mohammed Said to report on the Linant-Mougel plan. This Commission established decisively that there was no difference in level between the two seas.

In 1856 in Paris, Negrelli de Moldelbe formally proposed a definite plan for the construction of the Suez Canal. It became essentially the plan of the International Commission. It provided for a modification of the Linant-Mougel plan and the elimination of the locks; the cost of the work was estimated at 162 million gold francs. Negrelli was awarded the contract, but he died in 1858. The proposed canal section was 8 m (26.25 ft) in depth, 44 m (144.36 ft) in bed breadth, 80 m (262.47 ft) in water surface width, with 1 on 2 side slopes plus a berm of 0.6 m (about 2 ft) on each side at about 1/5 depth below the water surface.

Concurrently with the first issue of shares in 1958, a Supreme Works Council, composed of eight members, was formed in Paris. In August 1859, this organization decided to retain the depth of 8 m (26 ft 3 in), but to reduce the theoretical width at bottom to 22 m (72 ft 2 in), the width at the surface to 58 m (190 ft 3 in), and that the passing of vessels should take place at sidings situated at intervals of 10 kilometers (6-1/4 miles). These sidings would have an extra width of 5 m (16 ft 6 in) and be 300 m (328 yds) to 500 m (547 yds) in length. This was the 1859 plan.

The Canal was built in accordance with the 1859 plan with slight modifications at localities, stretching a length of 160 kilometers (99.42 miles), with side slopes 1 on 2 and a berm of 1.5 m (4.92 ft) on the west side and 2 m (6.56 ft) on the east side at about 1/5 depth below the water surface.

## The Canal Company, 1858 to 1956

At the end of 1858, de Lesseps founded the Compagnie Universelle du Canal Maritime de Suez. It has the triple character of (1) an Egyptian company with registered office in Alexandria, (2) a French joint-stock company incorporated under the French law with head office in Paris, and (3) international responsibilities in its functions. Because it was born of an audacious French engineering promoter's dream that such a canal could be built and the Frenchmen have the majority of shares, the Company has fittingly its Administrative Office in Paris. The canal zone has actually been owned by Egypt, while the Suez Company was merely enfranchised by Egyptian law to build, maintain, and operate the waterway under a 99-year concession from the date of completion of the work.

De Lesseps' objective to emphasize the international character of the Company was well indicated by the selection of members of the first Board of Directors which consisted of representatives of 14 countries which had shareholders. Though this tradition has been preserved, a more concentration of the distribution of board members has since naturally resulted. After the sale of Khedive's shares to Britain, there had been no Egyptian directors from 1876 to 1936. When, in 1936-7, the first agreements to bring about a closer association between the Conceding Power and the Concessionary Company were negotiated, Egypt became again represented on the Board by two members. The agreement of 7 March 1949 marked a further step by the immediate allocation to Egypt of 2 more seats on the Board, to which 3 more will be progressively added. Since 1875, Britishers have occupied board

seats only next to Frenchmen in number, because since then the British Government has owned 7/16th of the whole and also because British shipping has always contributed the dominant part to the Canal's traffic.

On the eve of the Nationalization of the Suez Canal Company in 1956, the Company had 32 directors comprising 16 Frenchmen, 9 Englishmen (3 from the government, 6 from shipping and business), 5 Egyptians, one American, and one Hollander, but actually there were only two from the British Government at that moment.

The reparticipation of Egypt in the governing body of the Company had been accompanied by increases in Egyptian personnel in recruiting staff. The agreements of 1936-7 had laid down the principle of a gradual integration of Egyptian employees into the Company. Their proportion was to reach 33 per cent in 1958, and, as a general rule, one vacancy in three was reserved to them. In fact, this proportion was reached in 1949.

The Convention of 7 March 1949 still further increased the rate of intake of Egyptian personnel, requiring the Company to engage 4 Egyptians for every 5 vacancies on the technical staff, 9 Egyptians for every 10 vacancies on the administrative staff, and 1 in 2 for captain-pilots and sea personnel. On signing the Convention, the Company proceeded to engage some 50 employees and to promote or engage 18 to fill intermediate and higher posts. Finally, it was agreed that the Commission set up by the Company to consider applications from Egyptian candidates should include a person of standing nominated by the Egyptian Government. By January 1956, Egyptian personnel rose to 53.6 per cent in technical and administrative staff combined, 16.8 per cent in captain-pilots, and 46.7 in sea personnel.

Thus, the history of the Company reveals in fact that it has been in turn Franco-Egyptian from 1858 to 1875, then Franco-British from 1875 to 1936, and Anglo-Franco-Egyptian since 1936 until it was nationalized in 1956. These characteristics are reflected in the distribution of its capital, in the composition of its Board and of its personnel, and in the nature and magnitude of the financial benefits accruing to the Egyptian Government.

## The Capital Structure and Financing, 1858-1956

At the beginning it had been estimated that the expenses would amount to 200 million francs gold (the franc of that time), and that was therefore the total capital of the Company. The total issue was divided into 400,000 shares at a par value of 500 francs gold. Ferdinand de Lesseps offered it on the French bourse in 1858. Wishing to stress the universal character of the Company from the very moment subscriptions to the capital were opened, he set aside blocks of shares for each of the nations interested in the success of the enterprise. The subscription list had been opened in the principal countries of the world, and the call was answered by applicants from 14 different countries besides Frenchmen and the Egyptian Government. The Frenchmen absorbed the large majority amounting to 52 per cent of the stock during the first month. Some countries abstained, and some did not take up the quota set aside for them. The French Government acquired no shares. None of Britishers was interested then, nor a Russian came forward. The Viceroy of Egypt, who not only granted the concession but also subscribed a block of 86,506 shares, stepping into the place of those who had declined to subscribe, took up their allotment, and became the holder of 44.15 per cent of the share

capital, thus bestowing on the financial plane a further pledge of that royal spirit of cooperation to which Ferdinand de Lesseps owned so much in the successful outcome of his enterprise. The subscriptions from the other 14 different countries together amounted only to the remaining 4 per cent of the shares.

The original estimated cost of 200 million gold francs soon proved insufficient. The engineers concerned had no precedents in canal building of this magnitude in the West at that time, though it had no comparison in magnitude with the Grand Canal built in China early in the 7th century. The work was carried out in conditions more difficult than had at first envisaged. Contingencies were mounting up. The outcome was that the expenses incurred by the actual work alone rose to 300 million francs, besides the company charges, the running costs, and various other expenses, which added up to about another 120 million francs. The total cost of the Suez Canal as it stood in 1869 amounted to 420 million francs of that period, which is equivalent to 100 million Egyptian pounds at the present day.

De Lesseps manifested his tolerant nature and his persistence in raising the necessary money to meet the mounting demand. The major part of it came from Egypt, partly in advance settlement of claims arbitrated by Napoleon III (1808-1873) in 1864 in favor of the Company, partly by purchase of the El Wadi estate and other odds and ends of superfluous Company property, and partly by Egypt's relinquishment of earnings during the first 25 years on the Khedive's 176,602 shares that Britain later bought. These compromises enabled the Company to float a loan of the value of the interest that Egypt was giving up.

There still remained a need for 100 million francs. De Lesseps arranged to raise that sum with a public loan. However, subscribers came forward only when he coupled the offer with a lottery that at once turned the loan popular.

Throughout the whole financing period, Egypt revealed an attitude that was vitally helpful. Her faith in de Lesseps withstood every strain. That country had known him well when he was only 27 years old, and by 1869, when the canal was opened to navigation, he was 64 years old.

The original 400,000 shares had a par value of 500 gold francs; they were split 2 for 1 in 1924 and now the 800,000 shares have a par value of 250 gold francs. The Canal has proved to be a successful commercial enterprise whose profits confirm de Lesseps' assurances to the original shareholders. The French families who put their faith in de Lesseps have held their shares closely. So has been England for her own shipping interests and canal profits. In 1928, a share of par value at 250 gold francs was worth on the bourse 18,000 francs. In 1955, the Canal's net profit amounted to about 30 million dollars. Up to 1956, as a result of the investment necessitated by the improvements carried out, the capital value of the Canal had more than tripled in relation to what it was in 1870. The capital return for Suez was very modest, as it did not exceed 5-1/2 per cent.

Though favorable as they were, the Suez Canal shares had followed far less favorable a course in terms of dividends than those of other French and American securities. Between 1929 and 1955, the Suez dividend expressed in dollars had slightly decreased, whilst the dividend index of variable income securities had increased in France (295 variable-income securities) by 46 per cent and in the United States (Dow-Jones industrials) by 63 per cent. However, it was not reflecting this situation on the "Bourse" that the Suez

quotation had dropped 40 per cent during this same period whilst the index of variable dividend securities had increased in France by 260 per cent and in the United States by 420 per cent. This depression of the "Suez" shares actually reflected the investors' fear of political dangers—and not the financial situation of the Company which had never been as sound as on the eve of the nationalization.

According to the Company's payments to the French Fiscal Authorities for stamp duty on securities, on the eve of nationalization in 1956, the Suez Canal shares were held by the owners in the following proportions:

| | | |
|---|---|---|
| British Government | 353,204 | shares, 44.15% |
| In Circulation | 446,796 | shares, 55.85% |
| Owned by French nationals (abt.) | 353,000 (+) | shares, (abt.) 79.00% |
| Owned by other nationals (abt.) (including hardly more than 10 Egyptians) | 93,800 (-) | shares, (abt.) 21.00% |

Among the French holders of Canal shares, there are 14,000 owners of registered securities and on deposit, and more than 80,000 owners of bearer shares.

In France, Suez Canal shares found favour from the outset with all classes of society—members of the profession, salaries employees, and tradesmen—which led the Company to be referred to in the British Parliamentary sarcasm as an "Association of Waiters." As time went on these securities have spread even more widely in such a way that the 353,000 or so shares held in France in 1956 were divided among more than 100,000 investors half of whom held no more than 2 shares each or in terms of value 140,000 francs on the basis of the average quotation of jouissance shares during the first half of 1956. This figure can be compared with the maximum of French Savings Banks deposits which was then 750,000 francs, showing that Suez Canal shares are in fact held largely by very small investors. In addition, French investors also hold 80,000 founders shares (owned by 15,000 holders), 35,000 debentures, and 422,000 "parts civiles de Suez" (owned by 50,000 holders). These various securities belong to nearly 170,000 French holders, in some cases "moral persons", but in most cases "physical persons"—of whom two-thirds hold from 1 to 5 shares, the average holding being 3 only. The capitalization of the various categories held in France represented more than 75 billion francs on the eve of nationalization.

As of December 31, 1956, on the basis of $1 = 350 francs, the Company had a total of assets equal to $382,662,348 of which $218,276,434 were in Egypt and $164,385,914 outside of Egypt. Besides capital shares, the Company had a liability of $8,723,161 consisting of 45,143 outstanding 3 per cent debentures, 2nd Series, of 500 gold francs each, and 8,255 Debentures of the same Series, called and remaining to be paid. Counting other liabilities of the Company, it had more assets than liabilities outside of Egypt.

## The Construction of the Initial Canal and Resort to Machinery, 1859 to 1869

On 25th April 1859, the first blow of a mattock was struck by Ferdinand de Lesseps, the builder of the Canal, to mark the beginning in the cutting of the Isthmus of Suez. It was struck on the shore of the ancient Pelusium where the town of Port Said now stands. Recalling that he headed the idea for the Suez Canal in 1832 at the age of 27, there had been an elapse of another 27 years before the actual commencement of excavating the canal.

A primary auxiliary undertaking was to convey drinking water from the Nile to the Isthmus by means of a fresh water canal. It was completed to the central part of the Isthmus early in 1862. The section to the South known as the Ismailia Canal was completed to Suez, except for the locks, on 29 December 1863. The conveyance of fresh water to Port Said in the North was accomplished by laying two conduit pipes, the first of which was put into service in 1864 and the second in 1866.

For the construction of the Maritime Canal, a total of 74 million cubic meters (97 million cu yd) of earth was excavated or dredged. The picks and baskets with which the first excavations were carried out were soon to be replaced by the most modern excavating and dredging equipment available at the time.

The agreement with Egypt called for her to provide what was termed "forced labor" on the canal. While the workmen's wages were higher than any pay they had previously earned, and they were also better housed and fed; a crisis, however, precipitated as the Porte said: "No more of this sort of thing" (forced labor). And the Porte's word was law. Forced labor on the canal was thus terminated.

It nevertheless turned out to be the salvation of the canal. There came the turning point that labor-saving machinery, including power dredges, had to be purchased or invested and put to use. De Lesseps farmed the work out in four separate contracts, and every one went well.

The most remarkable among the contractors was Alexandre Lavalley. Lavalley and Borel operated up to 60 dredgers at a time, of which a certain number were of a new type known as "long-arm" dredgers. Couvreux, another contractor, used excavators of his own invention.

Dredging proved to be cheaper than dry excavation. So Lavalley even transformed certain ground depressions into artificial lakes by flooding them from the fresh water canal, and dredging them where required, whilst at the same time gradually reducing the water level.

The construction of the jetties at Port Said offered another chief difficulty to be overcome, that is the distance from the quarries from which suitable stone for the bedding could be obtained. The use of stone from the quarries at Mex, near Alexandria, soon proved prohibitive in cost. It was then substituted by artificial blocks made of lime mortar, using the sand at Port Said as the basic material. These blocks lasted for more than 30 years until the beginning of the Twentieth Century when good quality stone could be taken from the Ataqa quarries on the western shore of Suez Roads.

On 18th March 1869, the waters of the Mediterranean entered the Bitter Lakes' depression. On 15th August of the same year, the waters of the Red Sea in their turn poured into these Lakes, thus at last effectively joining the two seas.

One of the most interesting events near the end of the initial construction was the filling of the Bitter Lakes. It took 5 months, from 1 May to 1 October 1869. The total cubic capacity of the lakes has been estimated at just under 1,500 million cubic meters (1,962 million cu yd), but a much greater volume of water had to be put in to allow for seepage and evaporation.

With the excavating tools and dredging equipment then available nearly a century ago, and owing to considerable difficulties encountered, including the climate, the desert region, and the epidemics, notably the cholera epidemic of 1865, it took 10 years 6 months and 22 days to complete the initial canal on 16 November 1869 ready for the first navigation. The work was, however, not

quite finished on that date, and at certain points the depths were not more than 5 m 20 cm (17 ft) or 5 m 50 cm (18 ft). There remained about 3 million cubic meters (approximately 4 million cubic yards) to be excavated.

Up to 31 December 1869, the total approximate cost of work was 287 million gold francs. The cost of excavation and dredging was just under 3 francs per cubic meter, which was treble the original estimate.

## Opening to Traffic in 1869 and Completion of Project in 1875

The Suez Canal was officially opened November 17-20, 1869, by the Khédive of Egypt, Ismail, and the Empress of the French, Eugénie, in the presence of Francis Joseph, Emperor of Austria, the Crown Prince of Prussia, the representatives of the leading powers, and a crowd of several thousand invited guests.

Among the vessels which took part in the ceremonies, the most important was the royal French ship Aigle, (Fig. 1), with Empress Eugénie and a gala court party aboard, starting from Port Said on the Mediterranean. The Aigle had a tonnage of 2,000 and her draught was only 4 m 60 cm (15 ft). The Company had forbidden ships having more than 5 m (16 ft 6 in) draught to participate in the inauguration. A fleet of sixty-eight commercial ships, all of them being sailing vessels in those days, started to go through the canal the next day. The era of steam mercantile ocean commerce had not yet dawned. Plying through the newly built canal traversing in an almost desert region under indifferent sail, the ships sluggishly moved forward (Fig. 2). The sailing from Port Said to Suez took some three days, the ships anchoring at dusk and resuming their course with the next sun.

The date November 17, 1869, marks the first day of the concession granted to the Canal Company by the Egyptian Government for a period of 99 years, which will end on November 16, 1968, the date at which the Suez Canal and all its dependencies will be freely handed over to the Egyptian State. Certain buildings with their annexes, the equipment, supplies, and dwelling houses will, however, be bought by the State at an agreed price or one fixed by experts.

Through the opening of the Suez Canal, the Eastern Mediterranean ports were reawakened and the movement of trade throughout all parts of the whole Mediterranean basin was enlivened.

The remaining work was continued as actively as possible without interfering with navigation, so as to reach the projected depth of 8 m (26 ft 3 in) throughout the Canal.

Concurrently, diverse complementary works were carried out, such as attaining the theoretical sections, rectifying the bends, enlarging the sidings, and digging basins at Port Said. The plans of the Supreme Works Council were not completely executed until 1875.

## Canal Maintenance, 1870 to 1956

Since the opening of the Canal to traffic in 1869, it has been necessary to constantly maintain the fairway by dredging. The passing of each ship through the Canal causes eddies which damage the banks. The materials thus worn away finally sink to the bottom of the Canal together with the wind-blown sand. The Canal would therefore gradually become unusable if regular maintenance dredging were not undertaken.

Fig. 1. The French Imperial Yacht Aigle at Port Said During Opening Ceremony in 1869

Fig. 2. Inaugural Convoy Passing El Gisr at the Time of Opening of Canal in 1869 (from a Water-Colour by Riou)

From 1884 to 1956, the maintenance works had been closely supervised by the Commission Consultative Internationale des Travaux whose organization will be described later in connection with improvement works.

Maintenance of the depths of the channel at Port Said against silting had constituted one of the most difficult technical tasks which the Company had to face constantly in order to keep the fairway from serious deteriorations. In February 1911, after a particularly heavy storm, 4 million cu yd of silt, equivalent to the normal silt over 5 years, were deposited in the channel in a few days. To solve this problem of maintaining the depths, systematic dredging and the extension of the western jetty at Port Said had been carried out simultaneously.

During World War II, all maintenance work had to be postponed on account of considerable reduction in traffic. When hostilities were over, a three-year restoration plan was drawn up at the end of 1945. Owing to difficulties in obtaining supplies during the post-war period, it took much more time than anticipated to finish the restoration work.

As of 1952, the dredging programmes were so arranged that depths would be kept not less than 11 m 70 cm (38 ft 4-1/2 in) at the lowest tides. This criterion depth was calculated to provide a safety margin of 1 m 34 cm (4 ft 6 in) under the keel of a transiting vessel drawing 10 m 36 cm (34 ft) which was the then authorized maximum draft. It was found that a large vessel steaming at the maximum authorized speed of 14 km per hr (7-1/2 knots) settles into the Canal to a depth of 90 cm (3 ft); there was thus provided a margin of 45 cm (18 in) between the bottom of the Canal and the lowest part of the ship's stern.

From 1870 to 1954, the total quantity of dredging for the maintenance of the navigation channel, the basins, the harbor of Port Said, and the maritime Canal amounted to 257,276,100 cu yd, which is 2.65 times the volume excavated during the original construction of the Canal.

## Continuous Deepening, Widening, and Other Improvements, 1876 to 1956

Conceived in the middle of 1850's when the biggest ships sailing regularly to the Far East were only a few thousand tons and had a maximum draught of only 6 or 7 m (20 to 22 ft), the Suez Canal, though opened to traffic in 1869, had not been fully completed until 1875 in accordance with the 1859 plan, providing a bed breadth of only 22 m (72 ft 2 in) and a depth of only 8 m (26 ft 3 in). This depth allowed the transit of ships with a draft up to 7 m 50 cm (24 ft 7 in) which was sufficient during 1870 to 1880. The rather narrow channel width caused nearly 3,000 groundings between 1870 and 1884. There could be no passing except at the Lakes or at widened lay-by stations, known as "gares", where a vessel could tie up while another passed. Up the Canal from Suez, ships had to wait until out-bound or south-bound traffic had cleared. Transit could only take place in daylight. It required an average sojourn time of 40 hr for a ship to transit the Canal, despite that there were only a daily average of 3 or 4 transiting vessels in the first ten years.

During the 86-year period from 1870 to 1955, transit requirements had gradually grown to the following proportions:

| | 1870 | 1955 |
|---|---|---|
| Average daily number of ships passing through the Canal | 1.33 | 40 |
| Maximum draught of ships that had passed through the Canal | 6 m 76 cm (22.2 ft) | 10 m 51 cm (34.5 ft) |
| Average gross tonnage of transit ships passing through the Canal | 1,350 tons | 10,500 tons |
| Maximum gross tonnage of ships that had passed through the Canal | 4,400 tons | 46,500 tons |
| Section of Canal should be | 350 sq m (3,770 sq ft) | 1,200 sq m (12,900 sq ft) |

Since 1876, to meet the demand of the era of steam mercantile ocean commerce and the ever increasing size of commercial and naval vessels, and to serve increasing traffic, the Canal Company had undertaken 8 improvement programs up to the time the Canal was taken over by Egypt in July 1956.

The first Programme of Improvements was planned in 1876. It consisted of (1) the lengthening of the sidings to 750 m (820 yds), then to 1,000 m (1,093 yds), (2) the building of new sidings, (3) the rectifying of several bends, (4) the widening of the Canal in the region of Suez, (5) the building of new basins in the ports, and finally (6) the protection of the banks by stone bedding. It was decided to execute the works at the rate of one million gold francs per year for 30 years. But soon it became obvious that this program was too modest to meet the traffic demands.

In 1884, there was set up the Commission Consultative Internationale des Travaux (the Suez Canal International Advisory Works Commission), which, for 72 years before the Canal was taken over by Egypt, had met annually in Paris to consider the schemes prepared by the Company's Technical Departments, and had thus closely supervised the improvement works. The Commission members were composed of most eminent engineers specializing in maritime works and experts in questions of navigation. Most of the members had been French and British engineers, but German, Austrian, Belgian, Egyptian, American, Spanish, Italian, Norwegian, and Dutch engineers had also served on the Commission at various times. In 1956, there were 18 members belonging to the above nationalities except German, Austrian, and Spanish.

The Second Programme of Improvements has been more commonly known as the 1884-5 Programme. It was to permit ships to pass one another without stopping. From 1886 onwards, the improvement works were actively pursued and the widening and deepening of the Canal produced an immediate improvement in navigation. Actual execution of the Second Programme of Improvements had been far less than the original project and only consisted of the following:

1887-1890, first deepening to 9 m (30 ft 2 in)
1887-1898, first widening to 37 m (121 ft 5 in)
1897-1900, restoration of previous sidings
1898-1904, deepening to 9 m (30 ft 2 in) and alteration to bends

These amounted to only 37,334,000 cu yd versus the originally projected 107+ million cu yd of excavation and dredging.

By 1901, it was realized that to meet the general tendency to increase the draught of ships it would be preferable to concentrate on deepening, rather than on widening the Canal. And it was further realized that the increasing in breadth of ships would preclude the feasibility of their passing one another in transit as embodied in the 1884-5 plan.

The Third Programme of Improvements was consequently drawn up in 1901 with the following principal features: (1) the deepening of the Canal to 9 m 50 cm (31 ft 2 in), (2) the reduction of the side slope of the submerged embankments to not more than 1 on 3, and (3) the provision of eleven new sidings 750 m (820 yd) in length and with an extra width of 15 m (49 ft 3 in); but in 1903 it was decided to increase the theoretical depth of the Canal to 10 m (32 ft 10 in) and in 1906 to 10 m 50 cm (34 ft 5 in). The new sidings were constructed during 1901 to 1904. From 1901 to the beginning of 1908, total excavation and dredging including new sidings amounted to 24,980,000 cu yd.

In 1908, the Fourth Programme of Improvements was decided to provide for the deepening of the Canal to 11 m (36 ft 1 in), and the widening of the channel to give a minimum width of 45 m (148 ft) at a depth of 10 m (32 ft 10 in). Previously, recesses were allowed at 3-mile (5-km.) interval on the Asiatic side. This enabled the widening by dredging the intermediate stretches. This programme as executed from 1909 to 1914 involved a total excavation and dredging of 28,252,000 cu yd.

From 1887 to 1911, the widening of the Canal in the region of Suez (a part of the First Programme) and other improvements at Port of Suez amounted to 1,177,000 cu yd.

The Fifth Programme of Improvements was drawn up in 1912 to attain a depth of 12 m (39 ft 4 in) and a width of 60 m (197 ft) at a depth of 10 m (32 ft 10 in) between the Bitter Lakes and Suez, and to provide a series of lesser widenings of the Canal to facilitate the passing of ships. This was executed from 1913 to 1924 with a total excavation and dredging of 39,892,000 cu yd.

Meanwhile, improvements to Port Said harbour up to 1919 involved a total excavation and dredging of 25,375,000 cu. yd.

In 1921, the Sixth Programme of Improvements was prepared to provide for works which were actually completed in 1934. The 1921 Programme was executed from 1925 to 1934 with a total excavation and dredging of 41,986,000 cu yd.

Between 1934 and 1948, only relatively minor improvement works were executed, except that, just before the outbreak of World War II in 1939, a wide siding was cut at Km. 22 and another a little to the north of El Qantara, amounting to 1,504,000 cu yd of excavation and dredging. However, from 1934 to 1951, the Canal waterway had remained in the same physical characteristics.

As the post-war period came in 1946, it became obvious that traffic was increasing to volumes distinctly above that of the pre-war period, accompanied by a tendency towards larger vessels. The increase in the breadth of ships and in the number of transiting ships, which had risen from a daily average of 10 in 1921 when the Sixth Programme was drawn up to 24 in 1948, had complicated the passing of ships. When two vessels coming from opposite directions passed each other in the Canal proper, one of them had to make fast to the Canal bank. The number of such moorings, which always require great skill, increases according to the square of the number of transiting ships, while the difficulty of manoeuvering becomes greater as the size of ships increases. It became necessary to group ships into convoys even at the risk of increasing

the waiting time for ships at the entrance of the Canal. Moreover, built through desert and swampland, the sea-level waterway cannot practically be widened further. So increased capacity was lately gained by "double-tracking" in the forum of bypasses, permitting two convoys of ships to pass without the delay of having one of them stop and anchor as is necessary in the single waterway to prevent collisions and bank erosion.

As the Great Bitter Lake provides a natural passing zone at one-third of the length of the Canal from Suez, the Seventh Programme of Improvements, as adopted at the end of 1948, consisted of (1) a general deepening of the Canal by 50 cm (1 ft 8 in) to allow the passage of ships with a draught of 36 ft as compared with the then permissible draft of 34 ft; (2) the digging, in the neighborhood of Ballah, at a point two-thirds of the distance from Suez to Port Said, of a by-pass canal approximately 12 kilometers (nearly 7.46 miles) in length; (3) the deepening from 10 m (32 ft 10 in) to 12 m (39 ft 4 in) of the tanker anchorages in Lake Timsah; and (4) the provision of enough new berths in Port Said harbour, to facilitate the grouping of convoys.

The Ballah Bypass was put into service on July 25, 1951 (Fig. 3). The enlargement of Port Said harbor was concluded on June 17, 1953, and the dredging along the axis to 14 m was completed in 1954. In all, the improvements carried under the Seventh Programme from 1949 to 1954, together with certain ancillary works, involved the removal of more than 40 million cubic meters (52,320,500 cu yd) of dredged and excavated material of which 370,000 cubic meters (484,000 cu yd) were of rock.

It is amazing to note that the above improvements from 1876 to 1954 were actually accompanied by a total excavation and dredging of 252,820,500 cu yd

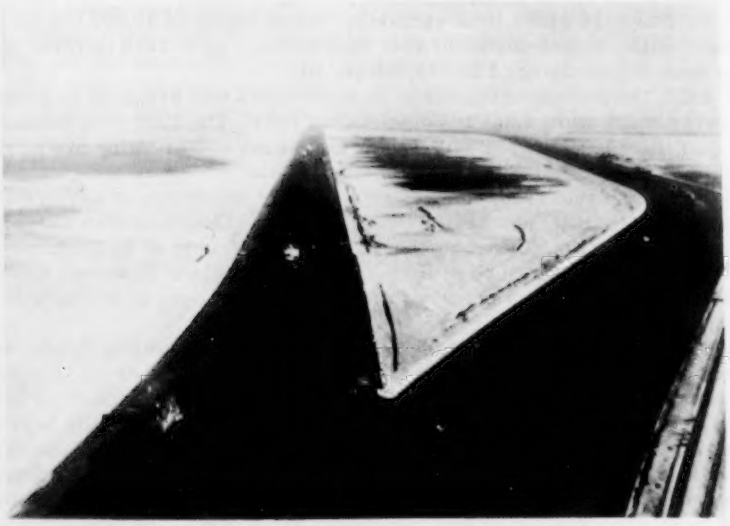


Fig. 3. Aerial View of Canal and Northern End of Bypass (Left) at El Ballah

which is 2.6 times that carried out for the original cutting of the Isthmus which amounted to only 97 million cu yd.

By 1954 the average daily number of ships passing through the Suez Canal rose to 36. According to information gathered by the Company, it was justified to make the forecast that within a few years the daily average total would be in the neighborhood of 45 ships and a daily maximum of 60 ships would have to be allowed. At the same time, the dimensions of the ships themselves had grown notably larger: tankers in transit with a main-frame of 160 sq m numbered 1,900 (16 per cent of total) in the year 1952, but rose to 2,700 (20 per cent of total) in 1954; and tankers in transit with a main-frame above 270 sq m was only 5 in 1952 but increased to 45 in 1954.

To meet the demand of such phenomenal growth of traffic, it was decided in 1954 to carry out the Eighth Programme of Improvements directed to increase in capacity and enlargement of the waterway to be finished in 1959. This programme was based on the probable character of the traffic about 1960, as far as it could then be estimated: 100 million tons of petroleum a year, a total of 17,000 ships a year of an average dead-weight of 23,000 tons. Because the El Ballah bypass had proved to be most successful, so two more were included in this improvement program which consists of:

(1) Digging the Port Fouad Bypass with a length of 2 km 200 m (1.37 miles), at the north end of the Canal, in which most of the ships will take up position one behind the other before entering the Canal (marshalling of southbound convoys). As soon as the last ship of a South convoy has left the way open, the first ship of the North convoy will be able to sail into the Canal, the other ships in transit following it in close formation. It will also shorten the stretch from Ballah Bypass to Port Said, which is the longest of the three segments that constitute the Canal.
(2) Digging the Kabret Bypass with a length of 7.3 km (4.54 miles) at the South end of the Great Bitter Lake, at about one-third point from the south end of the Canal.
(3) Deepening the Canal from 46.5 ft to 48.0 ft to allow ships drawing 36 ft to operate at normal speeds of 7 knots. With such depth, it is, however, more the width of the Canal that limits its capacity since vessels cannot pass one another freely; hence
(4) Widening the Canal to from 230 ft to 262 ft at the standard depth of 36 ft for measurement.
(5) Widening at lay-by stations to accommodate vessels which cannot proceed at convoy speeds and therefore have to tie up to permit convoys to pass.

In December 1954, the Company's Board of Directors voted the necessary credits for carrying out the digging of the bypasses, and the widening and deepening of the waterway in the South segment and in the El-Ferdan area.

In 1955, the Company spent a total of $8.6 million on these and other improvements. By the time the Canal was taken over by Egypt in August 1956, the two additional bypasses had been partly completed and half of the 8th Programme finished. All the improvement work was being done by a combination of Dutch, Belgium, and French contractors. This improvement program contemplated the removal of 98 million cu yd in 5 years as compared with the 95 million cu yd of excavation from the original canal which required a little more than ten years. With the necessary increase in equipment, the total expenditure would be approximately 50 million dollars.

All of the widening had been done on the east side of the canal, since the west bank is occupied by a railroad, a highway, and the so-called Sweet-Water Canal, which brings fresh water to the canal zone from the Nile near Cairo.

Except in the Lakes, the Canal is lined on either side by revetments to protect the banks against erosion due to wave wash from passing ships (Fig. 4 and Fig. 5). It had been gradually executed during the 80 years from 1876 to 1956. The majority are in quarry stone, some in steel sheet piling, some in huge concrete blocks, and some in still other types. The cost of replacing all the stone facings of the Canal banks would be about 5 million Egyptian pounds.

During World War II, the Company was compelled to agree to the building, by the military, of the swing bridge at El Ferdan for the duration of hostilities only. It was of the double span type built on two piers giving a clear opening of 60 m (196.85 ft) for the passage of ships. Two collisions of ships with the bridge had since occurred. After numerous representations by the Company, the bridge was replaced in October 1954 by another structure of different dimensions involving less danger to navigation. While the demolition of the old bridge was being considered, the second collision happened on December 31, 1954, when the tanker World Peace hit its western section, leaving one part of it resting on the ship's bridge while the other fell to the ground.

As of August 1956, the incessant improvements of the Canal through the past eight decades had resulted in the following physical dimensions:

| | | |
|---|---|---|
| **Lengths:** | | |
| Dredged channel in Port Said Roads from the lighthouse buoy to the Port Said lighthouse | 9 km | 5.59 mi |
| From Port Said lighthouse to Ismailia | 78 km | 48.47 mi |
| From Ismailia to the entrance channel in Suez Roads (15 km or 9.32 mi being in the Great Bitter Lake) | 83 km | 51.57 mi |
| Dredged channel in Suez Roads | 4 km | 2.49 mi |
| Total | 174 km | 108.12 mi |
| **Breadths:** | | |
| Typical at water level | 150 m | 492.12 ft |
| Minimum at water level more than | 120 m | 393.70 ft |
| Typical at theoretical intersections of side slopes with water level | 126 m | 413.00 ft |
| Between buoys | 75 m | 246.06 ft |
| Minimum under 10 m (32.81 ft) of water | 60 m | 196.85 ft |
| Theoretical at 10 m (32.81 ft) in the small Bitter Lake | 100 m | 328.08 ft |
| Typical at bottom | 42 m | 137.75 ft |
| Theoretical at bottom in muddy areas in northern section | 40 m | 131.23 ft |
| Depth along the axis | 14 m | 45.93 ft |
| Maximum draft for ships | 10.67 m | 35.00 ft |
| Cross-sectional area of water (in certain parts) | 1200 sq m | 12,917 sq ft |
| Projected draft for ships at end of 1957 | 11.0 m | 36.1 ft |

| | | |
|---|---|---|
| Projected minimum breadth under 11 m (36.09 ft) of water | 70 m to 80 m | 229.66 ft to 262.47 ft |
| Projected cross-sectional area of water after 1959 (5 times that of 1875) | 1,550 sq m | 16,684 sq ft |

In the curves and in certain other stretches, the breadth is greater. Some curves are 15 m (49.21 ft) wider. Standard side slopes are 1 on 3 on both sides, but there are variations from 1 on 1 to 1 on 4 at a few locations depending upon material. The submerged ledges or berms are of varying widths on each side.

With the exception of a dozen ships or so (the four largest ocean liners on the North Atlantic and several supertankers), all ships afloat in the world can clear the Suez Canal. Except in the case of two very large ships, it is possible anywhere in the Canal, so long as the bottom is not of a rocky nature, for one ship to make fast while the other steams past. However, two ships of exceptional dimensions can pass each other in Lake Timsah and the Bitter Lakes, or by making use of one or other of the two widened sections at km. 22 and 40, or by utilizing the bypass canal at Ballah.

Up to the time of Egypt's nationalization of the Canal in 1956, constant improvement and maintenance dredging during the previous 80 years had necessitated the removal of more than 450 million cu m (nearly 589 million cu yd) of spoil from the Canal, or 6 times as much as the volume originally excavated.

At the end of 1955, while the 8th Programme was still being carried out, the situation became obvious that giant tankers with a draft of more than 36 ft were increasing in number and the development of petroleum production in the Persian Gulf suddenly led to an expectation of a far more rapid increase in tanker traffic towards Europe and America than could be foreseen in 1954. Hence, new works were becoming necessary, and the prospect of a 9th Programme was not only envisaged but would also have to be started before the completion of the 8th Programme.

In the beginning of 1956, the Company got in touch with the International Chamber of Shipping and the principal American oil companies. In agreement with these representatives of the Users, the Company decided in the Spring of 1956 to entrust Ebasco Services Incorporated of New York with the complete study (to be finished in February 1957) of the traffic wishing to use the Canal up to about 1972. Without waiting for the conclusions of Ebasco's report, the Company had already, in the first half of 1956, undertaken the technical study of the works it expected to carry out in stages between 1958 and 1968 in order to develop the Canal according to the foreseeable needs envisioned at the beginning of July 1956 on the basis of a probable petroleum traffic of more than 150 million tons about 1965 and more than 200 million tons about 1970. At the latter date, it was estimated that there would be about 23,000 ships a year and that the maximum draft permitted in the Canal would have to be increased to 40 ft (about 12 m).

The study was carried out in view of a steady development of the Canal as a double waterway for the whole of its length—a waterway that would allow the transport of 400 million tons of petroleum a year. The portion which ought to be carried out by about 1970 would probably include the deepening and widening of only 60 kilometers of single waterway sections, as compared with 100 kilometers of double waterway allowing ships to pass each other.

Fig. 4. Bank Erosion Caused by Wash Due to the Passing of Ships

Fig. 5. Bank Revetment Between Km. 68 and 73

The expenditure involved by the projected 9th Programme of Improvements would probably be about five times as much as the 8th. The execution of the part corresponding to the traffic foreseeable in 1968, when the Company's concession expires, was intended to be met and could be met by the Company through expected receipts from a rapidly developing traffic.

## The Development of Canal Zone and Towns Since 1859

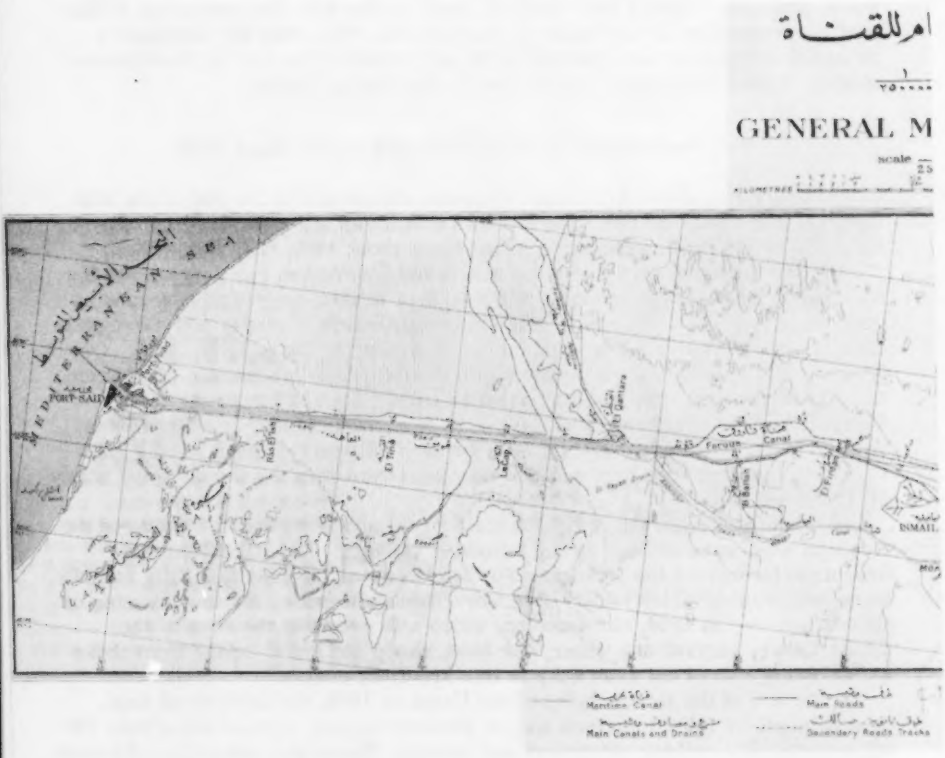
In the account of his Syrian and Egyptian campaigns at the end of the 18th century, Napoleon describes the Isthmus of Suez as the sixth desert of Egypt. Speaking of Suez in his memoirs dating from about 1835, Clot Bey writes: "This town, which consists of about a hundred houses and has some 3,500 inhabitants, is situated at the head of the Gulf of Arabia. ---- The surrounding country is arid in the extreme; there is no vegetation whatever and a complete lack of drinking water. The inhabitants have to bring water from the Fountains of Moses five leagues away on the other side of the bay. And even then it is brackish. Happily, our caravan had brought full goat-skins from Cairo." In 1840, the French Vice-Consul, Edmond Combes, described Suez thus: "---No trace of vegetation is to be seen outside the town; sterile sand covers the surrounding land and one may search in vain for the slightest shade or greenery."

As contained in the 5th series of his Journal and Documents, Ferdinand de Lesseps expresses himself in the following passage: "An expedition was organized to explore the Isthmus. For four persons, no less than fifty camels were required, of which twenty-five were laden with water for the crossing of this desert---- In 1854, our caravan, which was crossing the area of the Bitter Lakes, carried our water, our food, sheep and hens: apart from these animals there lived not even a fly in this appalling desert."

On the eve of the first digging of the Canal in 1859, the Isthmus of Suez, over a length of 160 kilometers and, in its central part, across more than 100 kilometers, was nothing but desert and swamp. There was not a sign of human life or of vegetable life all the way along the Isthmus except at its southern and northern ends. The little town of Suez, isolated and imprisoned within its arid surroundings and deprived of fresh drinking water, had but little more than 3,000 inhabitants. In the region of the ancient Pelusium, where Port Said was to rise, there were only a few fishermen's huts, while that which was to be Ismailia then appeared in the middle of the desert as nothing but a marsh surrounded by reeds, from which a traveller of the time, Barthélemy Saint Hilaire, was surprised and moved to hear the chirruping of some little birds.

Life made its appearance in the desert only after the fresh water was conveyed from the Nile, north of Cairo, by the navigable canals. It reached Ismailia in 1862 and Suez in 1863, and to Port Said by the first conduit pipe in 1864.

Besides the springing up of towns, the land has been brought under cultivation not only in the Isthmus but the whole way along the sweet water canals. From Cairo to Ismailia, along the Bitter Lakes, from Ismailia to Deversoir, and round Port Said and Ismailia, almost 66,000 feddans (approximately 69,190 acres or 28,000 hectares) had been won from the desert up to 1956. Despite this spectacular agricultural transformation 93 per cent of the population of the Isthmus is engaged in activities connected with the Canal traffic.




From the inception of the Canal work in 1859 the Company undertook the building of housing estates for its personnel and over 1,347 dwelling units had gone up before 1956, one-third for employees and two-thirds for workmen. They formed the nuclei around which fresh settlements and five Canal towns have sprung up.

The city of Port Said and the town of Ismailia were originally created by the Company. Port Said was conceived and built as an artificial port for the needs of the Canal and of local traffic. As soon as commercial life developed, they rapidly achieved a wide expansion. By virtue of the deeds of concession and the various conventions concluded with the Egyptian State, the maritime exploitation of Port Said had been an integral part of the Canal and was guaranteed by the Canal Company. Subject to the sovereign rights of Egypt, the Canal Company was responsible for its management and for the control of all its sea and commercial operations. Suez was a little trading and fishing port in 1850's. It has become an important industrial city where there are situated the Egyptian Government Petrol Refinery, the Shell Company Petrol Refinery, and a synthetic fertilizer plant.

Five principal towns on the canal are Port Said, Ismailia, and Suez on the west side, and Port Fouad (or Fuad) and Port Tewfik (or Taufiq) on the east

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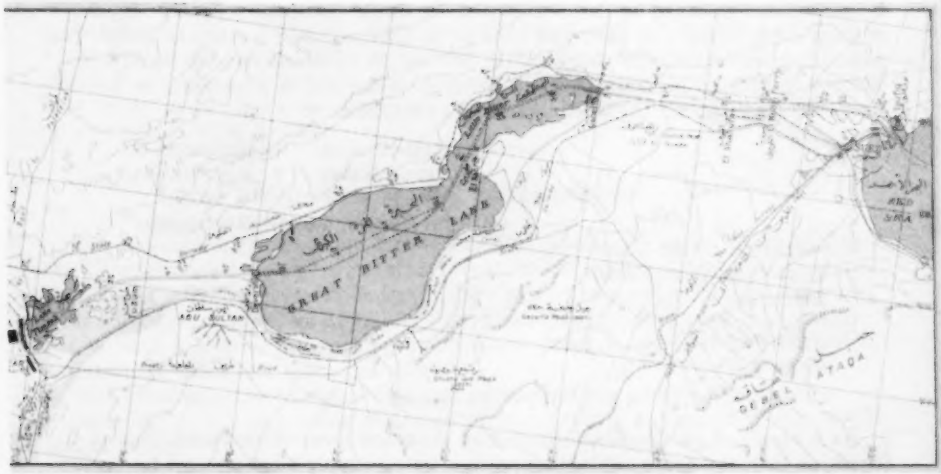


side. Port Said and Ismailia derived their names from the former heads of Egypt who ruled under the Turks. Viceroy Said Pasha gave de Lesseps the original concession when he organized the Canal Company in 1858. And Khédive Ismail, who later sold Egypt's stock in the Company to the British Government in 1875, was the successor of Viceroy Said in 1863. Fouad and Tewfik were later Egyptian rulers who gave their names to towns along the canal.

In 1902, in order to profit from its privileged geographical position at the crossways of the East and the West, Port Said was granted a Free Zone destined to be the principal center of redistribution in the Eastern Mediterranean for goods coming both from Europe and the Far East. In 1938, the Zone traffic amounted to 880,000 tons, but since World War II the rival Free Zones of Beirut and Genoa have considerably redirected the current of merchandise.

Central workshops of the Company were installed in Port Fouad on the Asian bank of the Canal opposite Port Said. They were opened by King Fouad in 1926.

The Company had built a new city and terminus at Port Fouad to relieve the waterfront congestion at Port Said, and had attempted to make it a model community for its purposes.

The agreement of 7 March 1949 between Egypt and the Canal Company provided for the establishment of a municipal authority in Ismailia to take over from the Company the administration of the various public services hitherto carried out by the latter.

Electric power is produced by two power stations located at Port Fouad and Port Said. The output had constantly been increased. It reached nearly 6-1/4 million kilowatts in 1950.

The growth of the five towns in the two Governorates (provinces) of the Isthmus of Suez can be best gauged by the consumption of filtered water furnished and ensured by three works of the Canal Company. It was only 2.7 million cubic meters in 1910 and rose to 22 million cubic meters in 1955. The population was only little more than 3,000 before 1859 and rose to nearly 420,000 in 1956. The distribution of filtered water and of population in the Governorates was for the year indicated as follows:

| | Distribution of Filtered Water 1954 (Million cubic meters) | Distribution of Population 1956 (Thousands) |
|---|---|---|
| Governorate of the Canal | | |
| Port Said, Port Fouad | 10 | 200 |
| Ismailia | 3 | 80 |
| Governorate of Suez | | |
| Suez, Port Tewfik | 7 | 140 |
| | 20 | 420 |

As a result of purification effected by slow and semi-slow Puech Chabal filtration process and by chlorination, the water supply is of perfect bacteriological quality. The remarkable degree of immunity which prevailed in the Isthmus during the cholera epidemic of 1947 testified this high quality. Laboratory control analyses numbered 6,300 in 1954.

The multiple functions of Port Said attained the following proportions in the 1950's. First, as the main station of the Canal, there were 25,945 ships of all sizes, excluding fishing boats, entering the port in 1954 and 14,666 ships of over 300 tons (Suez Canal gross tonnage) entering the port in 1955, and the maritime traffic attained an amount of 138,592,000 tons in 1954 for transit and coastal ships, which was ahead of all the ports in the world, even New York, at that time. Secondly, as a port of call, the total amount of goods and fuel handled in 1954 in the port—exports, imports, unloaded and embarked transit goods—attained 2,404,767 tons. Thirdly, as the second port in Egypt within a customs enclosure, it handled, in 1954, 626,402 tons of imported and exported local traffic, including 208,817 tons of British Army traffic. Fourthly, as a revictualling station for ships, the operation in 1954 accounted for 19,775 tons of coal and 1,358,322 tons of liquid fuel. After the turn of 1950's, more than 50 million cu ft of fresh water has been supplied annually to ships. Fifthly, as a Free Zone, there was 400,000 tons of Zone traffic in 1954. Lastly, as the most important fishing port in Egypt, besides the produce of seasonal sardine fishing, the local fleet landed more than 1,000 tons of fish a year, supplying 25 per cent of Egypt's annual fish consumption.

From 1952 to 1956, a special fishing harbor was built at Port Said by the Canal Company.

In the Canal towns, there were, in 1956, nearly 6,000 commercial and 3,000 industrial establishments. These towns are connected to each other by a railway and by two roads, the Treaty Road of the Government and the Canal Company's private road which runs along the Canal and the Lakes. The latter was preceded by the Ismailia-Port Said Railway which was constructed by the Company and later the equipment was handed over to the Egyptian Government; the old track linking the same two towns was transformed into the modern highway for heavy traffic and carried through to Suez. In the construction of the Treaty Road, the Canal Company contributed £E300,000 towards its cost.

Building in the Isthmus had been progressing at quite an exceptional rate. From practically nil in 1860, the number of buildings reached about 17,000 in 1947. From that time to 1952, entirely new districts had sprung up. At Port Said the price per square meter had risen from an average of £E2 in 1900 to an average of £E15 in 1949.

## The Development of Canal Equipment Since 1860's

Ever since the change from forced labor to machinery in the early 1860's, the Canal Company has gradually perfected bucket dredgers and improved the technique of dredging. In 1869, there were sixty dredgers available, each with a maximum power of 18 to 100 HP. Since then the Company has never had so high a number of dredging machines in service, but each individual one has become more and more powerful.

The trend of modernization of Canal equipment can be best seen from a comparison of average horse power of service craft at the end of World War II and in 1956:

| Service Craft | Average Horse Power of Service Craft | |
|---|---|---|
| | Equipment in Service in 1945 | Equipment Purchased since 1945 or under Construction up to Nationalization in 1956 |
| Dredgers | 600 | 5,200 |
| Tugs | 600 | 1,100 |
| Launches, Motor Boats | 60 | 115 |
| Total Available HP | 43,000 | 77,500 |

While half of the aggregate in 1956 consisted of new equipment, they represented, however, two-thirds of the available power.

In 1950, a 7,000-hp self-propelled suction hopper dredger with twin drag pipes was brought into use. This was named the Paul Solente after one of the Company's former chief engineers, and is one of the most modern machines of this type in the world. It was built at Nantes by the Ateliers et Chantiers de Bretagne. When actually working in the Port Said Channel it can dredge at least an average of 1,000 cu m (1,308 cu yd) an hr.

In 1955, the Dutch firm of shipbuilders, Werf Gusto at Schiedam, delivered to the Canal Company the self-discharging cutterhead hydraulic dredger, Louis Perrier, also named in memory of one of the Company's former chief engineers. This dredger was built at a cost of over $3 million. It has a potential power of 4,300 HP and is furnished with a pump capable of sucking up

material from a depth of 18 meters (59.05 ft) and of forcing it through floating and land pipes to a distance of 2-1/2 km (1.55 mi). This dredger, a real floating factory, can cope with 12,000 cu m (15,695 cu yd) a day.

Besides the huge dredgers, the Company had gradually acquired special purpose vessels, until it owned a big fleet of some 500 maintenance and salvage floating vessels of all kinds in 1956. They included four powerful tugs, the latest Dutch-built 4,500-hp Edgar Bonnet being one of the most powerful refloating tugs in the world, the Titan, the Atlas, and the Hercules, each developing 3,000 HP; two heavy refloating vessels, Castor and Pollux, each capable to raise a 2,000-ton ship; floating cranes from 8-ton capacity up including two of 150-ton and 80-ton capacity; huge American-built draglines for the Canal improvement program. The rest consisted of many tugs, tankers, lighters, cutters, ferryboats, pile drivers, and lifting vessels.

A 5,000-ton floating dock was brought into operation in 1952 and many stocks of different sizes made it possible to careen all the various vessels.

The Company's central workshops are at Port Fouad, equipped with more than 160 heavy machine tools, and supplied by a power station consisting of four 1,100-hp generators with the fifth nearing completion on the eve of nationalization. They employed a total of 1,400 to 1,500 workers in the power plants, workshops, and repair yards under the supervision of 16 engineers and 22 foremen. These central workshops are among the most important in the Middle East. An Apprenticeship Centre, the first ever set up in Egypt, maintained a four-year course including both practical and theoretical sections.

At the end of 1955, equipment in Egypt would, at cost, have totaled Frs. 11,636,420,007. Allowing for depreciation and a revaluation, the figure for this asset in 1956 became Frs. 10,740,047,125. At the time of nationalization, supplies represent an asset of Frs. 3,311,844,422.

Unfortunately, most of the Canal equipment were sunk or sabotaged in the Egyptian fighting early in November 1956.

## Tidal Variation, Currents, and Silt Since 1869

When the canal was opened to traffic in 1869, it had a bed breadth of only 72 ft 2 in (22 m). The tidal variation at Port Said was usually 1.44 ft (0.44 m) and reached 3.12 ft (0.95 m) at storm tide. On the Red Sea at Suez, the usual tide was between 2.6 and 4.9 ft (0.8 and 1.5 m) and storm tides reached 10.63 ft (3.24 m). Accordingly, a weak current up to 1.9 ft (0.6 m) per second developed in the stretch between Bitter Lakes north of Suez and Port Said.

In a paper presented to the International Navigation Congress held in Chicago in 1911, de Thierry presented interesting computations with reference to tidal currents in Suez and other canals showing agreement between computations and actual measurements. The variation of the cross-section and silting due to this current in the Suez Canal was negligible, and sanding up due to sand storms was of no importance. The choice of an open canal was thus very fortunate.

Since the widening of the channel during the Fifth Programme of Improvements was completed in 1924, the velocities of tidal currents have increased.

The Suez Canal is unique among the major canals of the world in the rapidity with which it could deteriorate under lack of maintenance. Hollowed out of sand dunes and swamps, which barely rise above sea level, its banks

are basically unstable and easily erodable under the wash of passing ships. Silt flowing in from the Mediterranean also requires constant maintenance dredging in normal times. Desert-blown sand along numerous stretches of the canal, though not too serious, constitutes another routine dredging task. Since the completion of the original canal, maintenance dredging from 1870 to 1954 has accounted for the removal of nearly 257 million cubic yards.

## The Evolution of Speeds and Capacity Since 1869

When the Canal was opened to traffic in November 1869, it took three days for the first sailing from Port Said to Suez. Beginning from that time until 1887, ships were only allowed to move through the canal during the day time. Between 1870 and 1884, transit ships had a total length of stay in the Canal from 38-1/2 to 46-1/2 hours and an actual navigating time from 17-1/2 to 18-1/2 hours. By 1887, ships with electric arc lights were permitted to move through at night. This concession cut the time of passage in half and almost doubled the capacity of the canal. During the 1900 to 1904 period, total length of stay was reduced to 18 hours and actual navigating time to 15-1/2 hours. Since 1930, total length of stay has been further reduced to from 13-1/2 to 15-1/2 hours and actual navigating time to nearly 11 hours.

The navigation velocity allowable in sea canals is dependent entirely upon the size of cross-section. Until the end of last century, for large vessels on the Suez Canal, the velocity at which ships were allowed to travel had been 5.40 knots (10 km per hour). According to the investigations made by de Thierry for the Suez Canal, it was shown that a ship with 28 feet (8.5 m) draft, in spite of the fact that the canal cross-section was practically seven times as large as the submerged ship cross-section, when traveling at a velocity of 8 miles (13 km) per hour, caused a lowering of the water level amounting to 4.49 feet (1.37 m). De Thierry, therefore, recommended that the ratio (n) of canal cross-section (F) to submerged ship cross-section (f) should not be smaller than 5. This relation, according to the late Professor Otto Franzius of Hanover, Germany, will be satisfactory for short canals, but for very long canals, carrying heavy traffic, a larger figure is desirable.

By 1900, the velocity was increased to 5.6 knots (10.4 km per hr) and by 1930 to 7.6 knots (14 km per hr).

Before 1939, when the number of ships averaged not more than 16 and only exceptionally went to around 30, ships could enter the Canal as they arrived, the Canal being wide enough to allow a moored ship to be passed by another ship sailing in the opposite direction. Beginning from 1948, the passage of ships had to be organized in convoys. Each day at fixed times two convoys leave Port Said and Port Tewfik (or Taufiq) and pass each other twice. Except at designated passing points, the other stretches of the Canal are traversed in one direction only, which is periodically changed. By 1950, the use of four convoys a day was approaching the limits of the Canal capacity.

In order to increase the capacity, the regulation speed for ships was raised, as from November, 1951, from 7.6 knots (14 km per hr) to 8.1 knots (15 km per hr), but it has not taken effect universally. The regulation speed of convoys from the North remains at 7.6 knots (14 km per hr). And as the large heavily laden tankers coming from the South could not maintain so high a speed, precise rules to speed up the flow of traffic by tightening up the convoys were put into force as from June 16, 1954, permitting the speed of

tankers from the South to be reduced to 6.5 knots (12 km per hr) on leaving the Great Bitter Lake as far as Km. 51, then at 7.0 knots (13 km per hr) to Port Said. About 1 h 15 min behind the last tanker, the rest of the ships pass through in a group at 7.6 knots (14 km per hr). The two groups thus arrive at Port Said without any break in continuity.

The existing capacity of the canal is about 42 to 43 ships a day, or 1,300 ships a month, and the bypasses would increase this to 50 ships a day. In fact, on December 13, 1950, there were 57 ships in transits. In 1955, the daily average number of transits reached 40 and the daily average in March of that year was 44.4. Whether the capacity is being kept pace with the growth of traffic can best be judged from the phenomenal increase of post-war annual tonnage figures for the canal. In 1946, 21.9 million tons of shipping passed through in twelve months. In 1951, it increased to 76.8 million tons; and, in 1955, it reached 107.5 million tons, of which 66.9 million tons consisted of petroleum products.

Thus tankers freighting oil have increased in recent years to more than 60 per cent of the tonnage through the canal and have afforded a corresponding proportion of its revenue from tonnage charges. Present tankers are generally of 25,000- to 40,000-ton class, and the prevailing tendency is toward larger tankers of 80,000- to 100,000-ton class.

Ships through the Suez Canal, however, have always been of lighter class than in the Atlantic. Navigation past Port Said for vessels up to 25-feet draft account for about 70 per cent of the traffic in normal times. In 1956, the passage of ships drawing 35 feet was authorized with certain restrictions. It would require too much power for vessels having deeper draft than 36 feet to move through the canal with its present width. Experimental studies on a small scale model conducted by Paul Blanquet in 1954 in the laboratory of the Etablissements Neyrpic at Grenoble, France, have shown that it is indispensable to widen the Canal at the same time that it is deepened.

When the second part of the eighth programme has been completed in 1959, the capacity of the Canal will be increased nearly four times since World War II to something like 180 ships a day.

## Chronological Bibliography on the Suez Canal, 1800 to 1957

In the foregoing sections of this paper, the important events associated with the historical development, organization, financing, construction, maintenance, improvements, and operation of the Suez Canal up to "nationalization" have been accounted for.

The nine volumes of de Lesseps' own works on the Suez Canal published between 1855 and 1881 contain about 2,000 printed pages. Altogether, so far as the writer is able to ascertain, from 1800 to 1957, there have been 106 published works, records, and literatures on the various phases of the Suez Canal, with 12 additional volumes for the more comprehensive works.

Instead of attempting to write any more exhaustive narrative which could not be complete with the maximum length allowable for the Society's Proceedings Papers, it becomes desirable to present, herein, the chronologically arranged bibliography to facilitate waterways engineers who are interested in specific phases of the Suez Canal to look into the original publications for full information. The chronicle as presented constitutes the principal narrative, while the chronological bibliography embraces the authoritative sources

heretofore published. Among the Canal Company's annual reports, only the last submitted by its Board of Directors to the 100th General Meeting of Shareholders held on June 25, 1957, is included in the bibliography.

The chronological bibliography from 1800 to 1957 follows:


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Paper 1771 September, 1958 WW 4

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

# THE AMERICAN LOCKS OF THE ST. LAWRENCE SEAWAY

John P. Davis,[1] A. M. ASCE
(Proc. Paper 1771)

## SYNOPSIS

The Eisenhower and Grass River Locks are the only two locks in the St. Lawrence Seaway that are being designed and built by the United States. These two locks are located in the International Rapids section of the St. Lawrence River near Massena, New York. Both locks are similar to other high lift locks and have filling and emptying systems that utilize longitudinal culverts, side ports and tainter type valves. A vertical lift gate has been provided at the entrance to the upstream lock to close off flow in the event of failure of the miter gates. The pool between the two locks is to be regulated by culverts placed in the lock walls. Development of hydraulic design criteria, selection of pool levels and a description of model tests that were made are presented in this paper.

## INTRODUCTION

This paper is presented to furnish members of the engineering profession general information on the Long Sault Canal Locks, including the hydraulic studies that were made, the criteria used and the development of various features of the two locks. The St. Lawrence Seaway comprises the natural channel and various improvements made along the St. Lawrence River that will enable ships to ply between the Great Lakes and the Atlantic Ocean. The distance along the St. Lawrence River from Lake Ontario to the City of Quebec, where the river enters the Gulf of St. Lawrence, is 342 miles. The total fall in this distance, the difference between Lake Ontario level and sea level,

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1771 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 4, September, 1958.

1. Chf., Hydr. Branch, U. S. Army Engr. Dist. Buffalo, Corps of Engrs., Buffalo, N. Y.

is about 246 feet. There will be a total of 7 locks between Lake Ontario and tidewater at the City of Quebec. The Long Sault Canal and locks are located in the International Rapids section which comprises a reach about 46 miles in length that extends from Ogdensburg, New York downstream to the point where the International Boundary leaves the river at the upper end of Lake St. Francis. The Long Sault Canal is approximately 10 miles in length; the Eisenhower Lock is located near the mid point of the Canal; and the Grass River Lock is located at the downstream end of the Canal, upstream from Cornwall Island.

## Dwight D. Eisenhower Lock

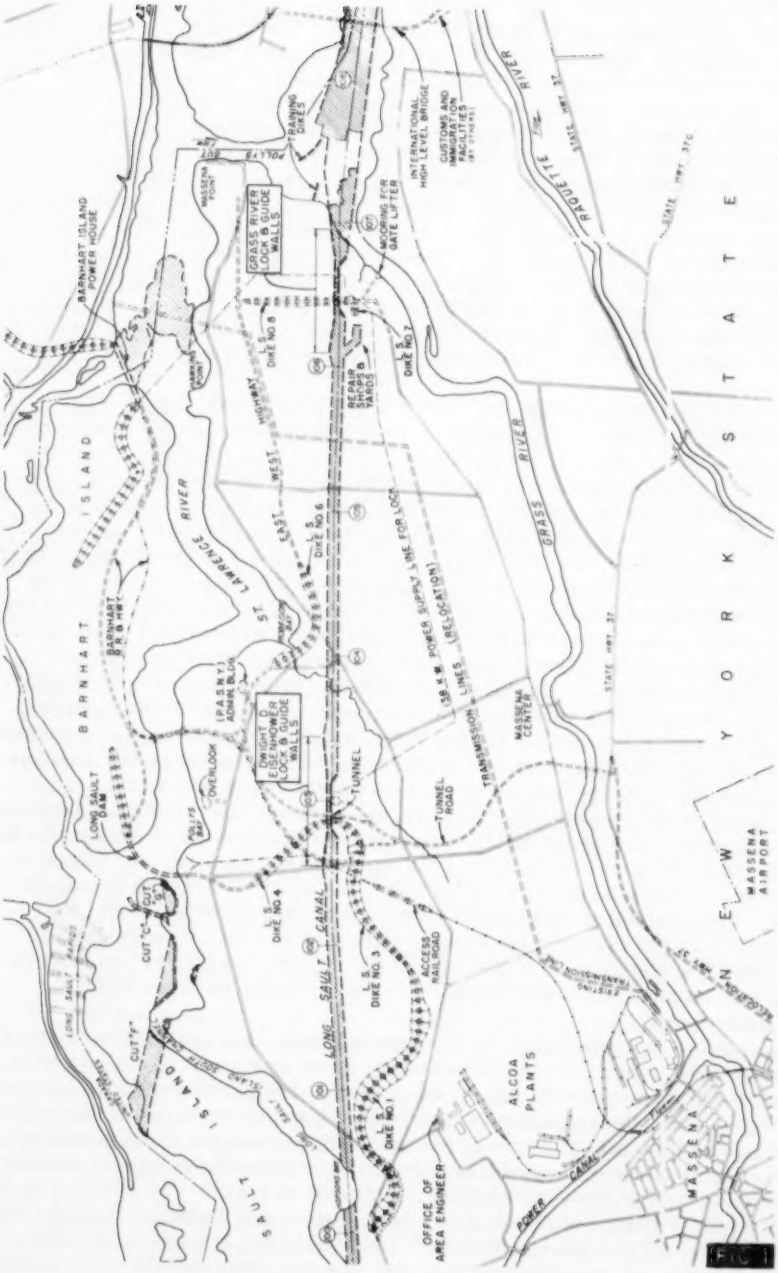
The Eisenhower Lock is located at the approximate mid point of the Long Sault Canal, about two and one half miles northeast of Massena, New York. It is the upstream lock of the two locks that are required to transfer vessels between the level of the power pool above Long Sault Dam and the river channel downstream from the dam south of Cornwall Island. Location of the Long Sault Canal, the Eisenhower and Grass River Locks and other features of the Seaway are shown on Fig. 1.

The Eisenhower Lock is 860 feet in length from gate pintle to gate pintle; 80 feet in width from face of wall to face of wall; and provides a minimum depth over the upper and lower gate sills of 30 feet. Miter gates are provided at each end of the lock chamber, and the normal usable length, which is the distance from the upper gate pintle to the fender boom in front of the lower gates, is 768 feet. The lift of the lock will ordinarily range from about 38 to 42 feet. The lock chamber is excavated down to rock which ranges from about elevation 140 to 144.

A vehicular tunnel is being built under the upper lock entrance to provide access to Barnhart Island Powerhouse and to the area that lies north of the Long Sault Canal. The tunnel is 29 feet wide. The roadway is 22 feet in width and has a vertical clearance of about 15 feet.

The upper pool at the Eisenhower Lock is to be regulated so that it will not exceed elevation 238 for a period of several years. After several years of operation, the upper pool may be raised, if agreed upon by the Governments of the United States and Canada, to range between a minimum elevation of 236 feet and a maximum elevation of 245 feet depending on the regulation of Lake Ontario. The elevation of 236 feet is the minimum mean monthly elevation at the upper lock entrance that is expected to occur under the regulation plan that will be adopted for Lake Ontario. Negative surges of as much as two feet can occur in the Long Sault Canal coincident with the minimum monthly elevation of 236. A minimum water surface elevation of 234 was adopted for design purposes, and with a required depth on the miter sill of 30 feet, the top of the sill was fixed at elevation 204 feet above mean sea level.

The pool in the canal between the locks will be controlled as nearly as possible at an elevation 200 feet above mean sea level, and will range between a minimum elevation of 197.5 and a maximum elevation of 202.5. The height of the lower miter gate at Grass River Lock is 85'-3" and was established by the lower sill elevation and the top of the lock wall of Grass River Lock. In order to permit use of identical miter gates at the lower end of both of the locks, the lower Lock sill at the Eisenhower Lock was fixed at elevation 166.5.


BARNHART ISLAND
BARNHART ISLAND POWER HOUSE
BARNHART BRIDGE
LAWRENCE RIVER
ST
LONG SAULT DAM
OVERLOOK
(P.A.S.N.Y.) ADMIN. BLDG.
DWIGHT D. EISENHOWER LOCK & GUIDE WALLS
GRASS RIVER LOCK & GUIDE WALLS
TUNNEL
TUNNEL ROAD
L.S. DIKE NO. 4
L.S. DIKE NO. 3
L.S. DIKE NO. 1
L.S. DIKE NO. 6
L.S. DIKE NO. 7
LONG SAULT CANAL
CUT "C"
CUT "B"
CUT "A"
LONG SAULT ISLAND
SAULT
ACCESS RAILROAD
OFFICE OF AREA ENGINEER
ALCOA PLANTS
POWER CANAL
MASSENA
MASSENA AIRPORT
MASSENA CENTER
TRANSMISSION LINES
38 K.V. POWER SUPPLY LINE FOR LOCK (RELOCATION)
WEST
EAST
HIGHWAY
REPAIR SHOPS & YARDS
MOORING FOR GATE LIFTER
TRAINING DIKES
INTERNATIONAL HIGH LEVEL BRIDGE
CUSTOMS AND IMMIGRATION FACILITIES
GRASSE RIVER
RAQUETTE RIVER
STATE HWY. 37C
STATE HWY. 37
NEW YORK STATE
FIG. 1

In the planning studies that were made prior to 1954, it had been considered necessary to provide a guard gate in the Long Sault Canal about midway between the Eisenhower Lock and Hopsons Bay. The guard gate was to have been provided to close off flow through the Eisenhower Lock in the event of failure of the miter gates. The structure, as planned, consisted of two concrete walls 110 feed apart equipped with sector gates which could be closed under head. During the later months of 1954 it was decided that it would be more economical to eliminate the guard gate structure and incorporate a vertical lift gate in the upper entrance to Eisenhower Lock to provide for emergency closure of the Canal. The gate was to be designed to rise vertically from a recess in the upper lock entrance immediately upstream from the miter gate sill, under conditions of free flow through the lock. In order to prevent flooding of the intermediate pool between the locks and overtopping of Grass River lock walls, the vertical lift gate would have to be designed so that it could completely close off flow in approximately 30 minutes. Hydraulic design studies that were made for the lift gate are described in subsequent paragraphs.

In the initial stages of planning of the Long Sault Canal locks it was decided that the general arrangement of the filling and emptying system would be similar to the systems used at other American high lift locks. The lock has a rectangular longitudinal culvert in each wall with wide ports that connect the culvert to the lock chamber. The intake manifold is located in the bottom of the lock entrance upstream from the vertical lift gate, and the discharge manifold is situated downstream from the lower miter gates. Each longitudinal culvert terminates in a 3-branch manifold that is connected to lateral diffuser culverts across the bottom of the lock immediately below the lower miter gates. The 6 lateral diffuser culverts have vertical ports in both the upstream and downstream sides. The filling valves are tainter valves and are located about 60 feet downstream from the upper miter gate pintles. The emptying valves, also tainter type, are about 65 feet upstream from the lower gate pintles. Design of the entire filling and emptying system was developed by model studies which are discussed elsewhere in this paper.

During the initial period of operation, when the upper pool at the Eisenhower Lock is maintained at a normal elevation 238, the lift at the Grass River Lock will be approximately 8 feet greater than the lift at the Eisenhower Lock. Since the lock chambers are the same length and width, considerably more water will be used for a lockage at Grass River than at the Eisenhower Lock. With continuous lockages at both locks at 30 minute intervals, the water required to maintain the intermediate pool was computed to be about 300 cubic feet per second. Also several successive lockages at Grass River Lock with no lockages at Eisenhower Lock could lower the intermediate pool appreciably.

In order to be certain that the intermediate pool level could be maintained satisfactorily, a separate culvert was built in the south wall of the lock to supply water to the intermediate pool. Probably the worst conditions, as far as lowering the intermediate pool is concerned, would result from 3 lockages at 30 minute intervals at Grass River lock with no lockages at the Eisenhower Lock. This number of lockages is believed to be about the maximum number that would occur without any coincident lockages at Eisenhower Lock. With 3 successive lockages at Grass River Lock and none at Eisenhower Lock, the volume lost from the intermediate pool would be about 218 acre-feet. The supply culvert is 5 feet wide by 7 feet high and will discharge 780 c.f.s. under

a 38-foot head. During three successive Grass River lockages, the volume discharged by the supply culvert would be about 100 acre-feet which would mean a net loss of 118 acre-feet. This loss in volume would cause the intermediate pool to drop only 0.4 foot. Therefore, the 5 foot by 7-foot culvert was considered to be adequate.

A by-pass or pool regulating culvert has been placed in the Grass River Lock which will prevent the intermediate pool from rising to an undesirable level if there should be several successive lockages through the Eisenhower Lock with no coincident lockage through the Grass River Lock. This pool regulating culvert is described later in this paper.

Recesses for stop logs have been provided in the lock walls upstream from the intake ports of the filling system and downstream from the lateral discharge culverts in the lower entrance so that the lock can be unwatered for maintenance and repairs. If the upper miter gates should fail and the vertical lift gate could not be operated, the upstream stop logs would be used as an emergency dam to close off flow through the lock.

A 4-foot square culvert has been placed in the north lock wall which runs downstream from the highway tunnel to a sump located in the downstream end of the north lock wall. This culvert is provided with valves so that it can be used to either conduct runoff that enters the highway tunnel into the sump or to drain water out of the lock chamber when the lock is to be unwatered. Two 9,000 g.p.m. pumps are provided in the sump which can unwater the lock in 24 hours. These two pumps have adequate capacity to pump the storm runoff that will drain into the highway tunnel. A 500 g.p.m. pump has been placed in the sump to handle seepage and minor drainage from various recesses in the lock wall that drain into the sump via the culvert.

## Filling and Emptying System

The filling and emptying system of the Eisenhower Lock was developed principally by means of model studies. In the initial design phases, filling and emptying times and maximum permissible hawser pull were selected through experience at other locks. The values that were used in preliminary considerations were:

a. Filling time not to exceed 10 minutes.
b. Emptying time not to exceed 10 minutes.
c. Hawser pull not to exceed 10 tons.
d. No excessive turbulence was to be permitted in the lock chamber, or in the entrances to the lock.

The general arrangement of the lock filling and emptying system was to be similar to other American high lift locks. Longitudinal culverts and tainter type valves were to be used. Filling and emptying ports in the walls were to be used if a satisfactory design could be developed. The culverts intakes were to be located in the bottom of the upstream lock entrance (upstream from the vertical lift gate) and the discharge ports were to be placed in manifolds across the bottom of the lower lock entrance downstream from the miter gates.

A general model of the complete lock was built to an undistorted scale of 1 to 24.24, and an additional model was set up to study and develop various types of filling and emptying ports. The models were built and the tests were

made by the St. Paul District of the Corps of Engineers at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota at Minneapolis, Minnesota. Tests were made on the lock model with three different sizes of culverts, two different intakes and two different arrangements of discharge manifolds. Six different designs of ports were developed and tested and it was found that the sixth one, type "F" gave the best results. Models of two different size lake steamers were built and were used in the lock model tests to determine the amount of hawser pulls that would be exerted on mooring lines during filling and emptying operations. The larger of the two models was a model of the Great Lakes Steamer "GEORGE M. HUMPHREY" which is 710 feet in length, has a beam of 76 feet, and has a maximum draft of 27 feet. The other steamer model was a model of the "GEORGE L. EATON" which draws 16.5 feet, is 253 feet in length and has a beam of 44 feet.

Numerous tests of the filling and emptying operations were made and hawser pulls, upper and lower pool levels, valve movement, water surface elevation in the lock chamber and time sequence were all recorded on a strip chart for each of the tests. Turbulence in the upper and lower entrances and in the lock chamber were observed visually during the tests and movement of confetti in the lock chamber during the tests was photographed. For each different model setup, a group of tests were run for 3 different lifts and for several times of valve opening.

The initial tests with Model 1 setup were started with culverts 12 feet wide by 14 feet high; culvert valve openings of the same dimensions; 14 lock chamber ports in each wall located approximately in the middle third of each lock wall and 96 discharge ports in 6 transverse culverts in the lower lock entrance. The first group of tests (Model 1 setup) showed that on the filling operations, with a 43-foot lift, opening the valves in 4 minutes would give a filling time of 8.8 minutes, including valve opening time, and hawser pull of 12 tons. On the emptying operation, opening the valves in 2 minutes gave an emptying time of 8.4 minutes and hawser pull of slightly over 10 tons. These tests were conducted with the model of the GEORGE M. HUMPHREY in the lock chamber. When the tests on Model 1 were completed it appeared that with slight changes in the culvert, filling and emptying times of 8 minutes and hawser pulls of not over 12 tons could probably be obtained. The 12 ton hawser pull was higher than the value originally selected, but was still not considered to be excessive. Accordingly, the values of 8 minutes for filling and emptying times and 12 ton hawser pull were adopted as design criteria.

The second group of tests were made with the model set up with culverts 15 feet high by 14 feet wide at the lock chamber ports. No other change was made in the culvert or in the valves, which were held the same as the setup used in the first group of tests. Two-foot extensions were added to the discharge ports of the discharge manifold which reduced turbulence in the lower lock entrance very effectively. The second group of tests (Model 2 setup) showed that on the filling operation, opening the valves in 4 minutes produced an 8 minute filling time with a 12 ton hawser pull and that on the emptying operation, opening the valves in 2 minutes gave an 8 minute emptying time with a hawser pull of 10 tons.

During the test runs on both models 1 and 2, vortices formed in the upper lock entrance over the intake port. This vortex action was considered undesirable and it was decided to redesign the intake entrance.

Since increasing the size of the culverts improved the filling and emptying conditions, it was decided that the model would be altered again and more tests

would be run. The culvert was enlarged to 15 feet wide by 16 feet high at the lock chamber ports; the intake ports were increased in size from 8 feet by 12 feet to 8 feet by 18 feet; and all other features of the model were left the same as in Model 2. This model setup was used in the third group of tests and was designated Model No. 3. Increasing the size of the culverts reduced the total filling time, including valve time, from 8 minutes to 7.7 minutes for a 4 minute valve time and also reduced the hawser pull to about 11 tons for a filling time of 8 minutes. Fig. 2 shows curves of filling time versus hawser pull for Model tests 1, 2, & 3.

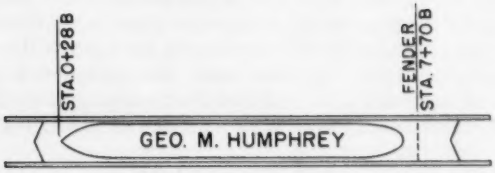
After the tests on Model 3 were completed it was decided to tentatively adopt the Model 3 design and proceed with testing for Grass River Lock. When the tests and design of the filling and emptying system for Grass River Lock were completed, that system was tested for the Eisenhower Lock and it proved to be a better design than the one that had been tentatively adopted. Therefore, the system that was developed for Grass River Lock (which is described later in this paper) was adopted for use on the Eisenhower Lock. This system utilizes the same culvert size that was used in the Model 3 tests but has 20 ports on each side of the lock instead of 14. On the filling operation, opening the valves in 4 minutes gives a total filling time (including the valve time) of 7.2 minutes and hawser pulls of 10 tons. On the emptying operation, opening the valves in 3 minutes gives an emptying time of 7.8 minutes, including the valve time, with hawser pulls of a little less than 10 tons. These results were adjudged to be very satisfactory and the 40-port design was adopted instead of the Model 3 design that had been tentatively adopted earlier.

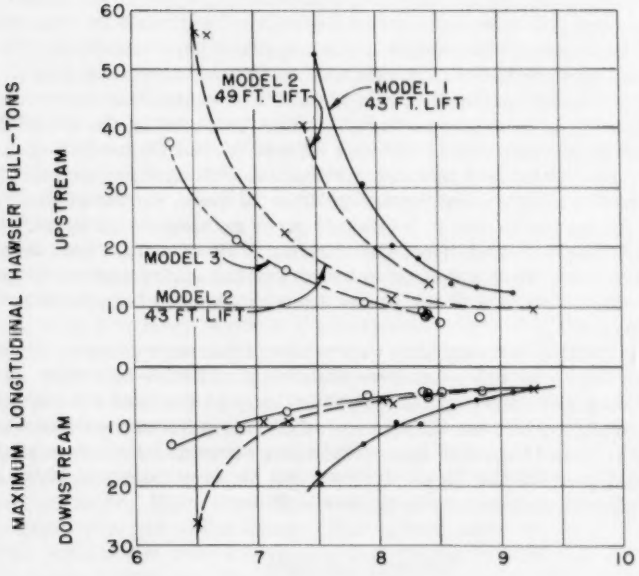
The culvert filling and emptying valves are of the segmental or reversed tainter type. This type valve has been found to give better hydraulic conditions during the filling and emptying cycle, particularly at medium and high lift locks, since it minimizes the entrapment of air from the valve recess into the culverts and results in quieter flow. The valve shape is based on model tests that were conducted for the lock valves for the McNary Lock and Dam, a Corps of Engineers project on the Columbia River.

## Vertical Lift Gate

Unprecedented problems were involved in all phases of design of the vertical lift gate. The gate was to be constructed with horizontal fabricated steel girders covered with a skin plate on the upstream face. As finally designed, it is approximately 83 feet wide, 46 feet high and has a maximum thickness of 10 feet. Before the gate could be designed it was necessary to determine: (1) the horizontal loading developed on the whole gate and on the individual girders; (2) vertical loading to which the gate is subjected during raising and lowering; (3) desirable crest shape; (4) whether or not negative pressures would develop under the nappe as the gate is raised; and (5) the rate of rise of the gate that would be required to keep the intermediate pool below elevation 202.5. Research on experience with lift gates and tests of lift gates in use, as well as theoretical considerations, indicated that subatmospheric pressure might develop on the downstream side of the gate as it was raised and seriously increase the horizontal loading. Because of the complexity of the loading problems and the difficulty in determining the loads analytically it was decided that studies would be made on an undistorted model of the gate at the St. Anthony Falls Hydraulic Laboratory, Minneapolis,

# FILLING TIME VS HAWSER PULL




NOTES:

ALL UNITS ARE PROTOTYPE
MODEL SCALE: 1 TO 24.24
CONSTANT VALVE OPENING RATE
SHIP IN LOCK:
NAME - GEO.M.HUMPHREY
DRAFT - 27 FEET
DISPLACEMENT-36,950 TONS
LOCK CHAMBER PORTS IN OPERATION:
4 THRU 17 EACH CULVERT
DUAL CULVERT OPERATION
UPPER POOL ELEV. 243.0
LOWER POOL ELEV. 200.0
DATA FROM TESTS 1-16 & 90-100 & 184-191

LEGEND

- MODEL 1; CULVERTS 12'X14' AT BOTH VALVES & LOCK CHAMBER PORTS.
- MODEL 2; CULVERTS BETWEEN VALVES ENLARGED TO 15'X 14'
- MODEL 3; CULVERTS BETWEEN VALVES ENLARGED TO 15'X 16' ENLARGED INTAKES

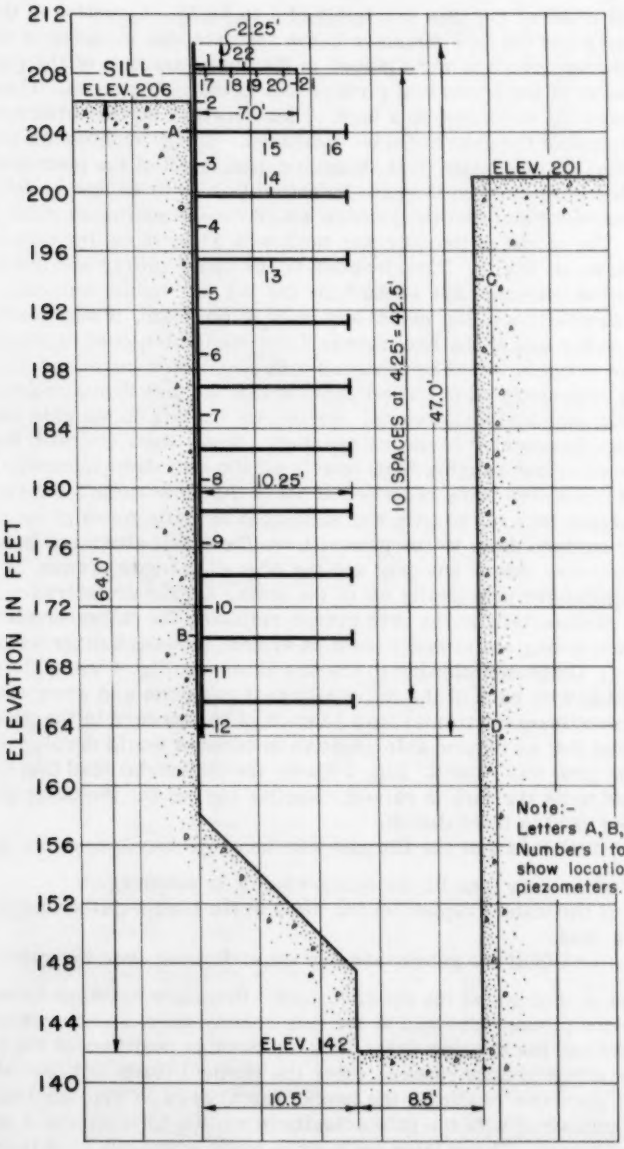
FIGURE 2

Minnesota. A model was built in a flume at the laboratory that reproduced 50 feet of the total width of the gate to a scale of 1 to 33.33. A portion of the upper lock entrance and the lock chamber below the gate was included in the model. Piezometer openings were placed in the upstream face of the gate, in webs of several of the horizontal girders and in the gate recess. These piezometers were all connected to a bank of manometers so that pressures at various points around the gate could be measured. There is shown on Fig. 3 a diagram of the lift gate model that shows location of all of the piezometers. Preliminary design studies on the gate indicated that 10 to 11 horizontal girders would be required and that the girders would have a maximum depth of about 10 feet. The model of the gate was built with 11 girders; the size and spacing are shown on Fig. 3. The elevation of the upper milter sill had been tentatively fixed at elevation 206 at the time the lift gate model was built. Subsequent to completion of the model and most of the tests, it was decided that the upper miter sill of the Eisenhower Lock would be placed at elevation 204 and that the lift gate would be designed with 10 girders instead of 11. No new tests were necessary as it was possible to use the test data already obtained by making suitable adjustments. Horizontal loading on the gate was determined from piezometer pressure readings. Tests were run with the gate at various positions ranging from nearly all the way down to nearly all the way up and pressures were recorded at all of the piezometers for each test. The upstream face of the gate was subjected to static water pressure from the water surface down to the gate sill. In the small clearance space between the upstream side of the gate and the edge of the gate recess, pressures were negative for practically all of the tests. On the downstream side of the gate the piezometers in the gate recess reflected the tailwater elevations and corresponding static pressures. A typical pressure diagram is shown on Fig. 4. Diagrams similar to the one shown on Fig. 4 were plotted for the tests made with each of the different crest positions and were used to determine the maximum horizontal load on each of the girders in the gate. The tests showed that no appreciable negative pressures would develop under the nappe as the gate was raised. Fig. 5 shows the horizontal load that Girder G-5 is subjected to as the gate is raised. Similar curves for the other girders were plotted and used in their design.

The total vertical load that the lift gate will be subjected to consists of:

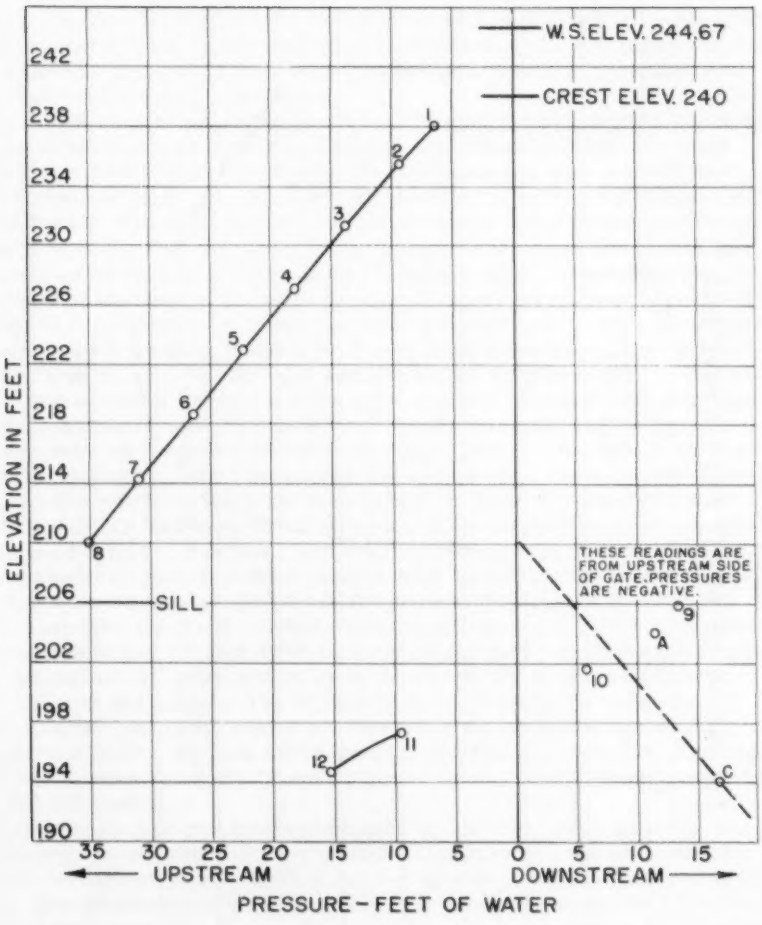
a. Weight of the gate less its buoyancy when it is submerged.
b. Weight of the water trapped on the webs of the horizontal girder.
c. Friction load.
d. Load on the top of the gate caused by water flowing over the gate.

This load has been designated the dynamic load. Pressure readings from the piezometers in the girder webs and in the gate recess were used to compute the dynamic load and the trapped water load for various positions of the gate from fully submerged to fully raised. After the vertical loads had been determined from the pressure readings, the model was altered so that vertical loads could be measured with the gate actually in motion by means of a dynamometer (Transducer). These later tests were made primarily to determine whether the force required to lift the gate would vary with different rates of raising the gate. Results of the dynamometer tests showed that raising the gate at different speeds would have no appreciable effect on the vertical load.




VERTICAL LIFT-GATE MODEL

FIGURE 3



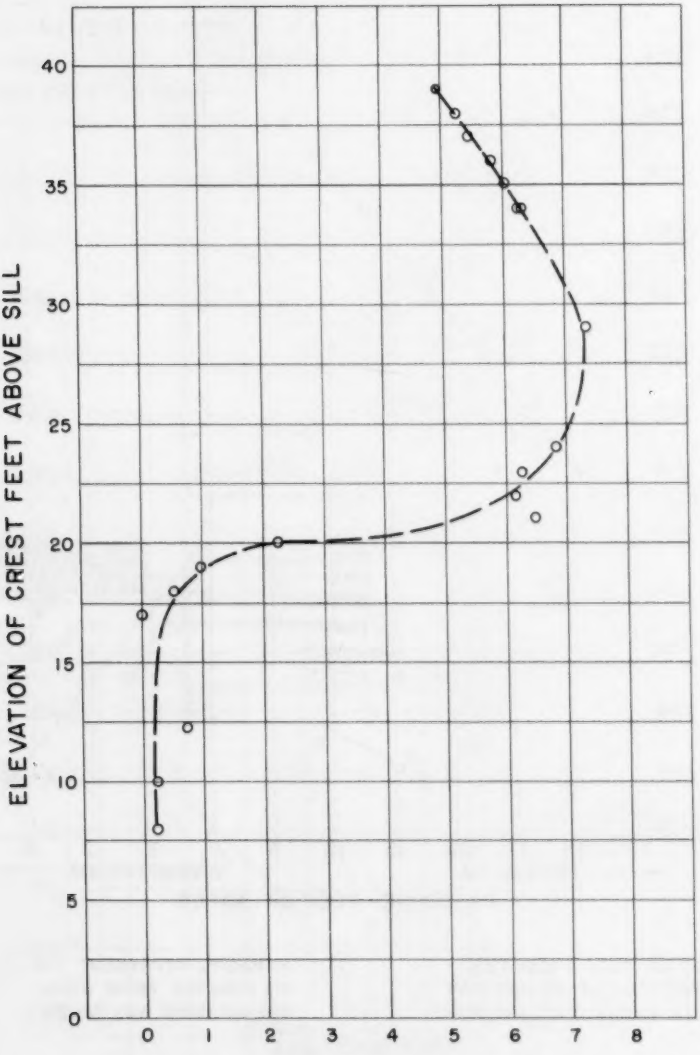

PLOT OF DATA FROM TEST CONDUCTED AT ST. ANTHONY FALLS HYDRAULIC LABOATORY.

NUMBERS REPRESENT THE PIEZOMETER FROM WHICH THE READING WAS TAKEN.

TEST CONDITIONS
MODEL 1
TEST 30
U.P. ELEVATION 245.03
L.G. ELEVATION 200.23
Qm = .510 cfs
Qp(50'GATE) = 3,340cfs

PRESSURE DIAGRAM

FIGURE 4




HORIZONTAL LOADING ON GIRDER G-5

FIGURE 5

The actual friction load to which the gate would be subjected could not be determined on the model as there was no means of simulating prototype friction conditions on the model. The friction load used in design of the gate was determined by the mechanical engineers that designed the lifting machinery. However, the friction load on the model gate was determined and was used in computing the dynamic load.

The gate's buoyant weight is constant as long as it remains fully submerged. The model tests showed that the gate remained submerged until the crest reached elevation 226, at which point the gate became a free overfall weir with an aerated nappe. At elevation 226, the overfalling sheet of water sprang free of the top of the gate entirely and there was no water falling on the top girder or running down the back side of the gate. At this elevation, the gate lost some of its buoyancy and continued to lose buoyancy throughout the remainder of its rise. Also, when the gate becomes a free overfall weir with an aerated nappe, water is trapped on each of the girders as they emerge or rise above the tailwater. Holes have been placed in the webs of the girders of the prototype gate so that this water will drain as the gate rises. In the tests, as the gate continued its rise and the crest reached elevation 239, the nappe no longer sprang free of the top of the gate. From elevation 239 throughout the remainder of the gate's travel (to elevation 245) the overfalling sheet of water fell on the web of the top girder and spilled down the back side of the gate. As the water ran down the back side of the gate, it flowed onto webs of all of the girders that were out of water and filled all of them to the depth of the girder flange or 10 inches. The drain holes in the girder webs were not large enough and cannot be made large enough to prevent the 10-inch depth of water from accumulating on all of the girders that are not submerged. Thus, when the gate crest reached elevation 239, there was a trapped water load equivalent to a 10-inch depth of water on the entire area of each girder web that was out of water, exclusive of the top girder, which amounted to 34,000 pounds per girder. The number of girders out of the water for any position of the gate crest varied with the water surface elevation on the downstream side of the gate. With the gate crest at elevation 239, 6 girders were out of water exclusive of the top girder and the total trapped water load was 204,360 pounds.

The dynamic load was first determined by pressure readings from the piezometers in the web of the top girder. The maximum dynamic load was found to be equivalent to a depth of 3.1 feet of water over the entire area of the girder web and occurred when the gate crest reach elevation 239. The total vertical load on the top girder, determined by this method, was 99,600 pounds or 1,245 pounds per linear foot of gate for an 80-foot length of gate.

In the tests that were made with the dynamometer, the gross load or the force required to lift the gate under free flow conditions was measured. The dynamic load was determined by subtracting the sum of the friction load, trapped water load and buoyant weight from the gross load. The vertical friction load on the gate at any position was computed by means of a friction coefficient and the horizontal load on the gate. The friction coefficient was developed by dividing vertical load by horizontal load at successive gate positions as the gate moved from elevation 204 to 226. Over this part of the gate's travel it was fully submerged. Its buoyant weight was not changing; there was no trapped water load; and the dynamic load was inappreciable. Thus, virtually all of the vertical load between elevations 204 and 226 was friction load and buoyant weight which made possible determination of an

average value for a friction coefficient for the model. The maximum dynamic load occurred when the gate reached elevation 239 in both the pressure tests and the dynamometer tests. However, the dynamometer tests showed a total dynamic load of 2,000 pounds per foot of gate (80-foot gate) which was considerably greater than the 1,245 pounds per foot that was obtained in the pressure tests. It was recognized that the dynamic load developed from the dynamometer tests were subject to any errors that were inherent in determining the friction load in the model. But, the dynamometer tests could not be ignored, consequently the maximum dynamic load of 2,000 pounds per foot of gate (80-foot gate) was adopted for design purposes and is considered to be on Girder No. 1.

Since the model tests showed that no appreciable negative pressures would develop under the overfalling sheet of water when the flow over the gate became aerated, no air vents were considered necessary. The vertical recesses in the lock wall at the ends of the gate supply all of the air needed. Aeration holes have been provided in the vertical diaphragms between the girder webs so that air drawn down through the wall recesses can pass through the entire area behind the skin plate of the gate.

The lift gate is designed primarily for emergency use and will not be required to pass ice while being raised, or to discharge water into the lock chamber. Therefore, it was not considered necessary to provide a curved or ogee shaped spillway crest for the gate.

In the event of failure of the miter gates, water would flow through the lock into the intermediate pool and would soon overtop the lock walls at Grass River Lock. A special test on the lift gate model showed that with the upper pool at elevation 245, the maximum discharge through the lock would be 60,700 c.f.s. The rate at which the gate would have to be raised to prevent the intermediate pool from exceeding elevation 202.5 was determined by consideration of the storage in the intermediate pool, the inflow through Eisenhower Lock, and the outflow that could be discharged through the filling culverts at Grass River Lock. It was assumed that if such a catastrophe should occur it would take 10 minutes to mobilize to raise the gate and that the emptying valves at Grass River Lock would be open in five minutes. With an average outflow of 12,500 c.f.s. through the Grass River Lock culverts, it was found that if the lift gate was raised in 19 minutes (total time from miter gate failure to closure of about 30 minutes) the intermediate pool would not exceed elevation 202.3.

## Grass River Lock

The Grass River Lock is located at the downstream end of the Long Sault Canal, about 3 miles downstream from the Eisenhower Lock. It has the same dimensions as the Eisenhower Lock, i.e. 860 feet in length from gate pintle to gate pintle; 80 feet in width; and a minimum depth over the miter sills of 30 feet. Miter gates, culvert valves and most of the design features are similar for both of the locks. Grass River Lock differs from Eisenhower Lock principally because Grass River Lock has no vertical lift gate or vehicular tunnel; the lift is greater; and the lock chamber floor at Grass River Lock is higher in relation to the filling and emptying culverts and ports than it is at the Eisenhower Lock. The bottom of the lock chamber is at about elevation 106 (top of rock) and the lift will usually range from about 45 to 49

feet. As stated in the foregoing paragraphs, the upper pool at Grass River Lock will be controlled as nearly as possible at elevation 200 and will vary between a minimum elevation of 197.5 and a maximum elevation of 202.5. The lower pool at Grass River Lock will range from a minimum elevation of 150 to a maximum elevation of 157. The maximum elevation of 157 was based on model studies made in connection with design of the Cornwall Island channel improvements and the minimum elevation was established by consideration of the effects of wind action during extremely low flow conditions in the river. With a minimum lower pool elevation of 150 and a required depth on the miter sills of 30 feet, the elevation of the lower sill was established at elevation 120. The top of the lock wall is at elevation 205, which made the lower miter gates 85 feet high. As noted earlier in this paper, it was considered desirable to use identical miter gates at the lower entrance to both locks. The top of the upper miter gate sill at Grass River Lock was established at elevation 165.5. Selection of this elevation was partially governed by the fact that spare gates are to be provided that can be assembled in sections so that they can be used at either of the locks.

A regulating system has been provided at Grass River Lock so that the intermediate pool can be prevented from exceeding the permissible maximum elevation of 202.5. The area of the intermediate pool plus the adjacent land area that drains to the pool totals about 5.4 square miles. The runoff from storm rainfull over this area must be by-passed to the St. Lawrence River in order to prevent the intermediate pool from rising to undesirable levels.

The regulating system consists of an overflow weir 40 feet long located in the back side of the upstream guard wall which discharges into a culvert running through the South Lock wall and through the lower guard wall to a discharge port in the end of the guard wall. The crest of the overflow weir has been placed at elevation 200.0, and will have a submerged entrance to minimize the possibility of ice and debris getting into the culvert. Design of the overflow weir was based on the estimated peak rate of runoff from the 15-year all season storm. Runoff hydrographs for separate portions of the area draining to the intermediate pool were combined into an inflow hydrograph into the pool and this hydrograph was routed through storage for several different lengths of weir to determine the maximum pool elevation and discharge. With a weir length of 40 feet, the maximum pool elevation would be 201 and the maximum discharge would be 125 c.f.s. The 50-year all season storm was also investigated and when routed through the intermediate pool produced a maximum pool level of 201.4 and a peak discharge of 170 c.f.s.

The unwatering system at Grass River Lock is generally similar to the system at the Eisenhower Lock. The pump pit is located in the downstream end of the North wall and has two 9,000 g.p.m. pumps and one 500 g.p.m. pump.

## Filling and Emptying System

Design of the filling and emptying system for Grass River Lock was developed through model studies in the same manner that the system for Eisenhower Lock was developed. Criteria on filling and emptying times and on hawser pulls that were used in the tests on the Eisenhower Lock were used initially for Grass River Lock tests. The lock model at the St. Anthony Falls Hydraulic Laboratory was altered to simulate conditions at Grass River

Lock and tests were initiated by using the same culvert, intake, ports and discharge manifold that was adopted for the Eisenhower Lock. The lift is greater and the lock chamber floor is higher, relative to the culvert, at Grass River Lock than at the Eisenhower Lock. Because of these factors the turbulence and hawser pulls in the first group of tests were far too great. In further tests, various schemes were tried which included:

a. A lower lock chamber floor.
b. Extensions on the lock chamber ports.
c. Larger ports.
d. Different alignments of ports.
e. A lower culvert section at the lock chamber ports.
f. A greater number of ports.

Considerable difficulty was experienced in developing a system that would give satisfactory conditions in the lock chamber. It was also found that vortex action over the intake was far worse than had been experienced in the Eisenhower Lock tests, and the intakes were redesigned.

Testing was pursued until an arrangement of the ports and culverts was developed that gave satisfactory results. Many tests were made of variations in port size and arrangement, however reasonably complete testing was done six different schemes or model set-ups. The system, as designed, has 20 ports on each side of the lock chamber which are equally spaced over a distance of about 420 feet in the middle section of the lock. The culverts are 15 feet wide by 16 feet high at the ports and are 12 feet wide by 14 feet high at the valves. The ports are 4 feet high by 3 feet, 1 inch wide at the throat and the bottom slopes toward the lock chamber. The axis of the 10 downstream ports are normal to the lock wall and the 10 upstream ports are inclined upstream at an angle of 3.5 degrees. The lock chamber floor was held at the elevation of rock, which was about elevation 106.

Final results of the testing showed that on the filling operations, use of a 4 minute valve opening time would give a filling time of 8.4 minutes (including valve opening time) and a maximum hawser pull of 10 tons. On the emptying operation, a minute valve opening time gave an emptying time of 8.3 minutes and a hawser pull of less than 10 tons. Turbulence in the lock chamber was very minor and the system was considered to be satisfactory. The ports and culverts are being built according to the final design developed on the model. A complete report of the model testing is under preparation by the St. Paul District of the Corps of Engineers and will be available at a later date.

## ACKNOWLEDGMENTS

This paper has been based on the studies that were made by the Corps of Engineers in their work on the St. Lawrence Seaway. Particular acknowledgment is given to staff members of the Corps of Engineers in the Buffalo District, the St. Paul District, the North Central Division and the Office, Chief of Engineers.

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Proceedings of the American Society of Civil Engineers

# SCIENTIFIC DEVELOPMENTS IN RIVER TRANSPORTATION[a]

Clarence Reuben Horton, Jr.[1]
(Proc. Paper 1772)

## SYNOPSIS

The paper describes progress made in model testing of push type river towboats and barge fleets. Previous methods are shown to be inaccurate due to scale and neglect of propulsion effects. Progress in improving efficiency of river towboats by the new model techniques is evaluated in terms of performance and economics. A major testing program is continuing using a new model basin in Holland.

The most comprehensive model testing program in the history of western river transportation was started in 1953. It began on a rather large scale and is continuing at an ever increasing rate. It is not yet time to report all the details, but so many important and revealing general facts have been obtained that this interim report is being given for the benefit of those doing similar work, as well as for the operators of river towing vessels or those studying our waterway system.

Tests were actually started in 1950 at the U. S. Navy Model Basin at Carderock, Maryland. Large scale, self-propelled model tests of higher speed types of barges and boats were carried out in varying water depths, combined with flow studies in the circulating water basin and free maneuvering tests in the turning basin. It was immediately apparent that previous methods of estimating performance from ordinary model resistance tests of barges and measured performance of full size towboats gave results that were completely false in the high speed region. These methods had given useful answers in the very low speed region, say 5 miles per hour; (at least the service allowances that had been necessary were no larger than the spread in

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1772 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 4, September, 1958.

a. Presented at ASCE Convention in Chicago, Ill., February 25, 1958.

1. Chf., Marine Engr., Dravo Corp., Pittsburgh, Pa.

reported service performance). Another conclusion from these first tests was that the Kort Nozzle could give very worth while gains at the highest speeds ever likely to be reached in river towing.

Unfortunately, it was also clear that a continuation of the extensive program which was now needed for predicting the service performance of all modern types of river equipment would require more funds and time than anyone dared to think about.

In 1953, by a very happy combination of circumstances, model testing on a large scale was resumed at the Netherlands Ship Model Basin at Wageningen, Netherlands. This establishment, an essentially commercial laboratory, had excellent facilities for the purpose, and had tank time, outstanding personnel, and sufficiently low cost to make the work feasible when it developed that three major interests[2] were eager to combine their efforts. Testing has been continuous since then, divided between the various programs which have been introduced as we increased our confidence in the method and the value of the work.

There is nothing new about the method of testing being used. It had in fact been used by Capt. McEntee in 1925 in his studies for the Inland Waterways Corporation, which confirmed the value of the screw propelled towboat and spoon bowed barges. It was used because many years of testing of sea-going ships had established the method. It is simply the exact duplication in miniature of the whole tow, complete with propulsion, with sufficient size to avoid serious scale effects on propellers and frictional resistance. However, there were problems with Western River push towing; for example the enormous size of the tow compared with the diameter of the propellers, and the scarcity of tanks large enough to use large scales for modern tows. 1370 ft. long x 140 ft. beam, full size, reduced to 101.5 ft. long by 10.4 ft. beam is the largest model so far, yet model propeller diameters were only 8 to 9 inches. There were many other special problems, but the wide experience of Wageningen with shallow draft and canal vessels of European types solved or eliminated all of them, with one exception, of which more later.

One of the first jobs at Wageningen was to evaluate the methods of the past. Chart 1 shows the result of one series of tests to study scale effect, and the influence of including or omitting the towboat and propulsion effects when trying to determine the actual push required for a fleet of barges:

Curve A is the result of a small scale resistance test without towboat. Curve B is the same test run on the larger scale used at Wageningen. Curve C is the model prediction of the actual push which would be required in operation, including the increase in push caused by the pumping effect of the hull on the resistance of the fleet ahead of it. It is obvious from this chart that any pretense to accuracy requires <u>at least</u> the scale used at Wageningen and self-propelled testing.

It was necessary to obtain data for comparison of model tests with full size performance, and since most of the reliable full size tests had been run with a dynamometer barge 100' x 26' between the towboat and the barge fleet, it was decided to run the model tests shown on Chart 2. This shows that the insertion of the dynamometer barge increases the push required, which has some logic except that the magnitude is very great, yet slightly <u>less</u> power is

2. Mississippi Valley Barge Line, Union Barge Line, and Dravo Corp. Engineering Works Division.

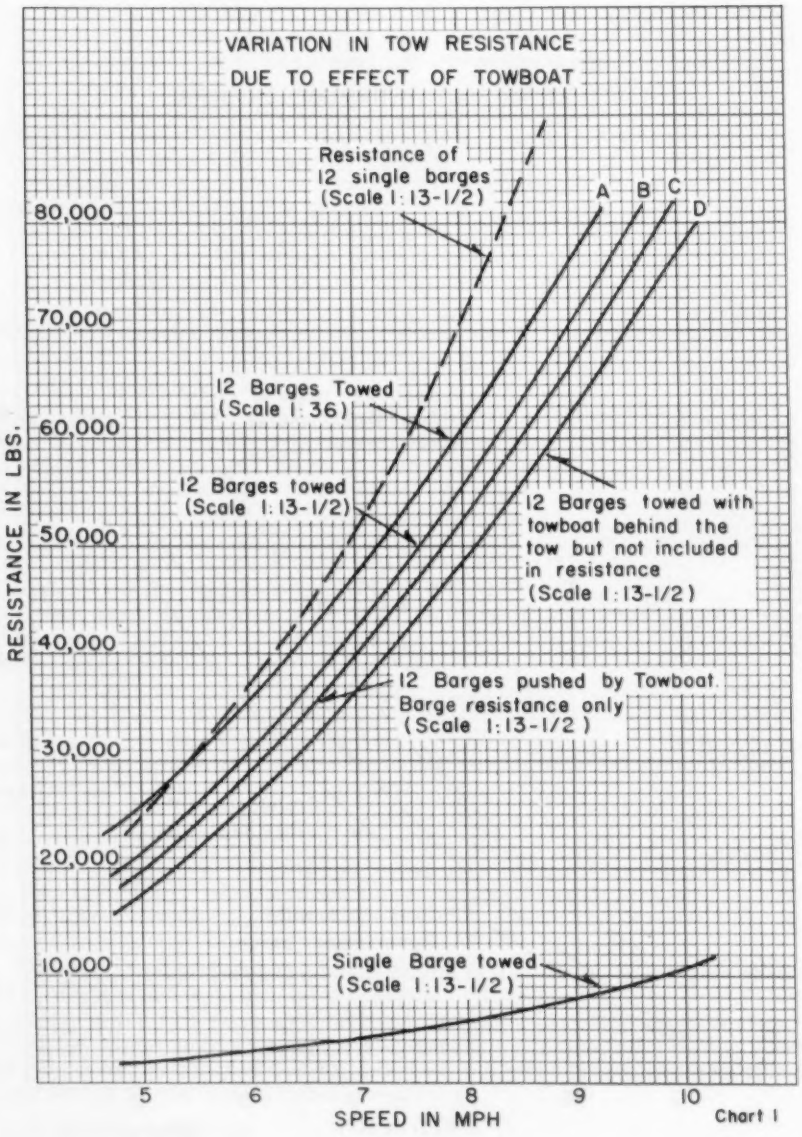
VARIATION IN TOW RESISTANCE
DUE TO EFFECT OF TOWBOAT
Resistance of
12 single barges
(Scale 1:13-1/2)
A
B
C
D
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
12 Barges Towed
(Scale 1:36)
12 Barges towed
(Scale 1:13-1/2)
12 Barges towed with
towboat behind the
tow but not included
in resistance
(Scale 1:13-1/2)
12 Barges pushed by Towboat.
Barge resistance only
(Scale 1:13-1/2)
Single Barge towed
(Scale 1:13-1/2)
RESISTANCE IN LBS.
5
6
7
8
9
10
SPEED IN MPH

Chart 1

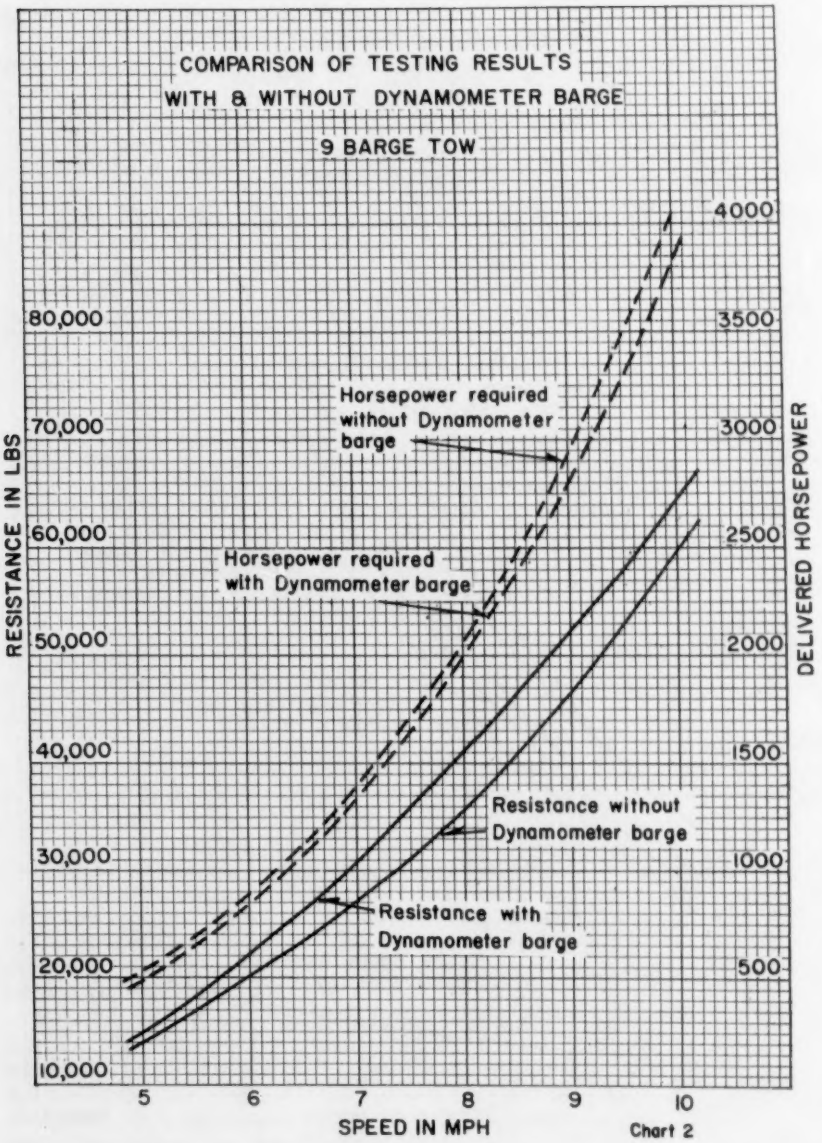
COMPARISON OF TESTING RESULTS
WITH & WITHOUT DYNAMOMETER BARGE
9 BARGE TOW
RESISTANCE IN LBS
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
DELIVERED HORSEPOWER
4000
3500
3000
2500
2000
1500
1000
500
Horsepower required without Dynamometer barge
Horsepower required with Dynamometer barge
Resistance without Dynamometer barge
Resistance with Dynamometer barge
5
6
7
8
9
10
SPEED IN MPH

Chart 2

required to make the same speed. It was gratifying to see that the dynamometer barge had not seriously affected the overall performance of the tow, but what was the explanation for making better speed with higher resistance? (Actually the total resistance effect is not that shown, but the "integration" of the towboat and its fleet is a very complex flow situation, and resistance transfers between towboat hull and barges as the situation is changed.)

As the testing progressed, we accumulated a large number of results for the same towboat behind different fleets. These values, for identical horsepower and all in deep water, are shown on Chart 3. A few spots of this sort which had been used in the past to determine a single line of push versus speed for each towboat design, and this line had been combined with towed resistance results, like Curve A on Chart 1, to predict a speed. It is obvious that the performance of a towboat cannot be represented by a single push-speed line, that comparisons of the push produced by towboats are meaningless unless the tows are identical, and that applying the observed push from one test to a different towing condition without further information is just guesswork. Chart 3 also shows the line of push versus speed for the 9 barge fleet which we use in all towboat tests.

All of the conclusions which invalidated the past methods were rather discouraging especially remembering the files full of expensive barge and towboat model tests which now appeared useless for making accurate commercial predictions. Fortunately, this has not been a serious matter because many designs used in those older tests have become obsolete. Testing for the future is being conducted on a sound basis and it has been possible to develop correction factors for old model tests in a few cases.

The designs that were made obsolete were made so in part by the program described here. It was apparent around 1950 that the rapid upward trend of relative labor and equipment costs was going to require revised equipment and operating methods. Two typical situations that could be projected for mass transportation on the Lower Mississippi are shown on Charts 4 and 5. In 1950, the largest boats in this trade were of about 2500 equivalent H.P. on the scale shown. The need was for boats of greatly increased effective output, without increase in draft, but with increased tow handling power for larger tows and, of course, of improved efficiency if possible.

The value of the current model testing program cannot be demonstrated more clearly than by showing Chart 6. This chart shows the results obtained by about two years of designing and testing and re-designing, culminating in construction of four vessels of the improved type shown by the upper curves. The middle curve represents the best previous Kort Nozzle towboat, and the lower a typical open screw towboat.

This improved performance was obtained by a series of changes in design, some of which had been considered economically impractical, some of which had been considered wrong, and some of which grew out of the tests themselves. It has been amazing how much designs could be improved when factual answers could be obtained quickly. Perhaps the scope of the program will be clear from a partial list of items investigated:

1. Propeller diameter.
2. Propeller pitch ratio.
3. Propeller pitch distribution, area, number of blades, blade section, etc.
4. Steering rudder location, shape, angle and forces.
5. Flanking rudder location, shape, angle and forces.

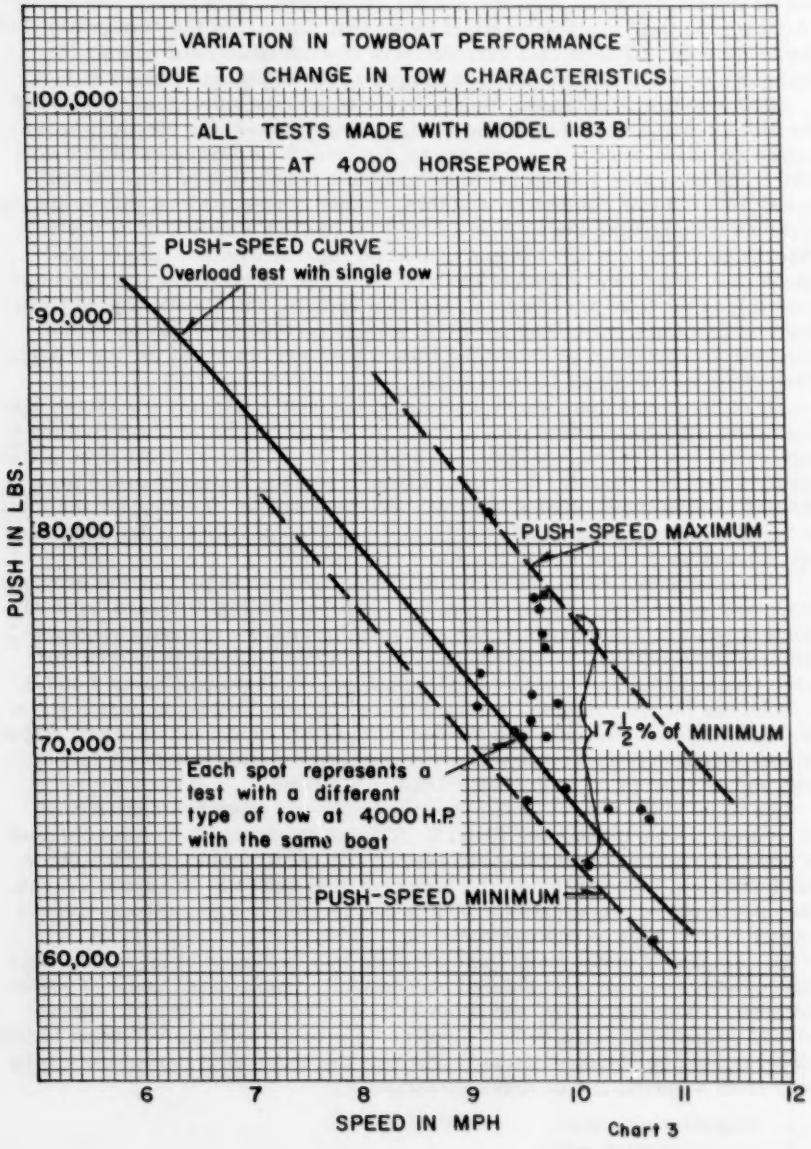
VARIATION IN TOWBOAT PERFORMANCE
DUE TO CHANGE IN TOW CHARACTERISTICS
ALL TESTS MADE WITH MODEL 1183 B
AT 4000 HORSEPOWER
PUSH-SPEED CURVE
Overload test with single tow
PUSH-SPEED MAXIMUM
17½% of MINIMUM
Each spot represents a test with a different type of tow at 4000 H.P. with the same boat
PUSH-SPEED MINIMUM
100,000
90,000
80,000
70,000
60,000
PUSH IN LBS.
6
7
8
9
10
11
12
SPEED IN MPH

Chart 3

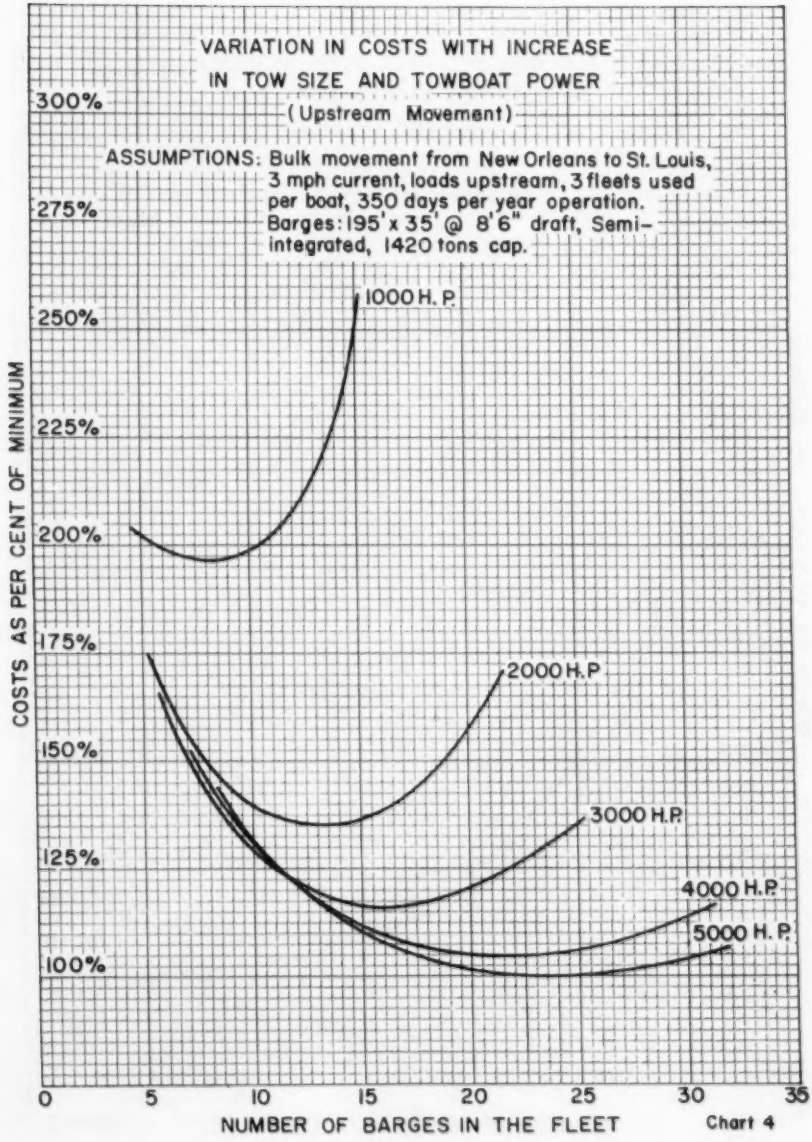
VARIATION IN COSTS WITH INCREASE
IN TOW SIZE AND TOWBOAT POWER
(Upstream Movement)
ASSUMPTIONS: Bulk movement from New Orleans to St. Louis, 3 mph current, loads upstream, 3 fleets used per boat, 350 days per year operation. Barges: 195' x 35' @ 8'6" draft, Semi-integrated, 1420 tons cap.
COSTS AS PER CENT OF MINIMUM
300%
275%
250%
225%
200%
175%
150%
125%
100%
1000 H. P.
2000 H.P
3000 H.P.
4000 H.P.
5000 H. P.
0
5
10
15
20
25
30
35
NUMBER OF BARGES IN THE FLEET

Chart 4

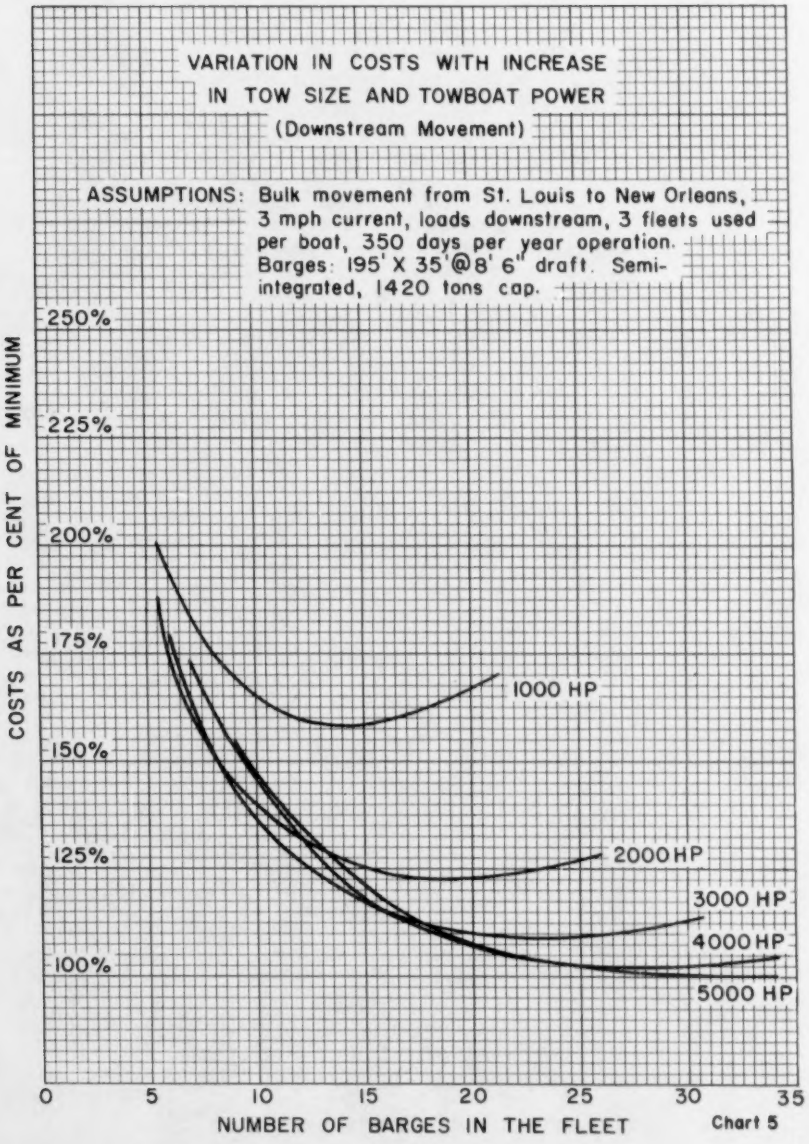
VARIATION IN COSTS WITH INCREASE
IN TOW SIZE AND TOWBOAT POWER
(Downstream Movement)
ASSUMPTIONS: Bulk movement from St. Louis to New Orleans, 3 mph current, loads downstream, 3 fleets used per boat, 350 days per year operation. Barges: 195' X 35'@8' 6" draft. Semi-integrated, 1420 tons cap.
COSTS AS PER CENT OF MINIMUM
250%
225%
200%
175%
150%
125%
100%
1000 HP
2000 HP
3000 HP
4000 HP
5000 HP
0
5
10
15
20
25
30
35
NUMBER OF BARGES IN THE FLEET

Chart 5

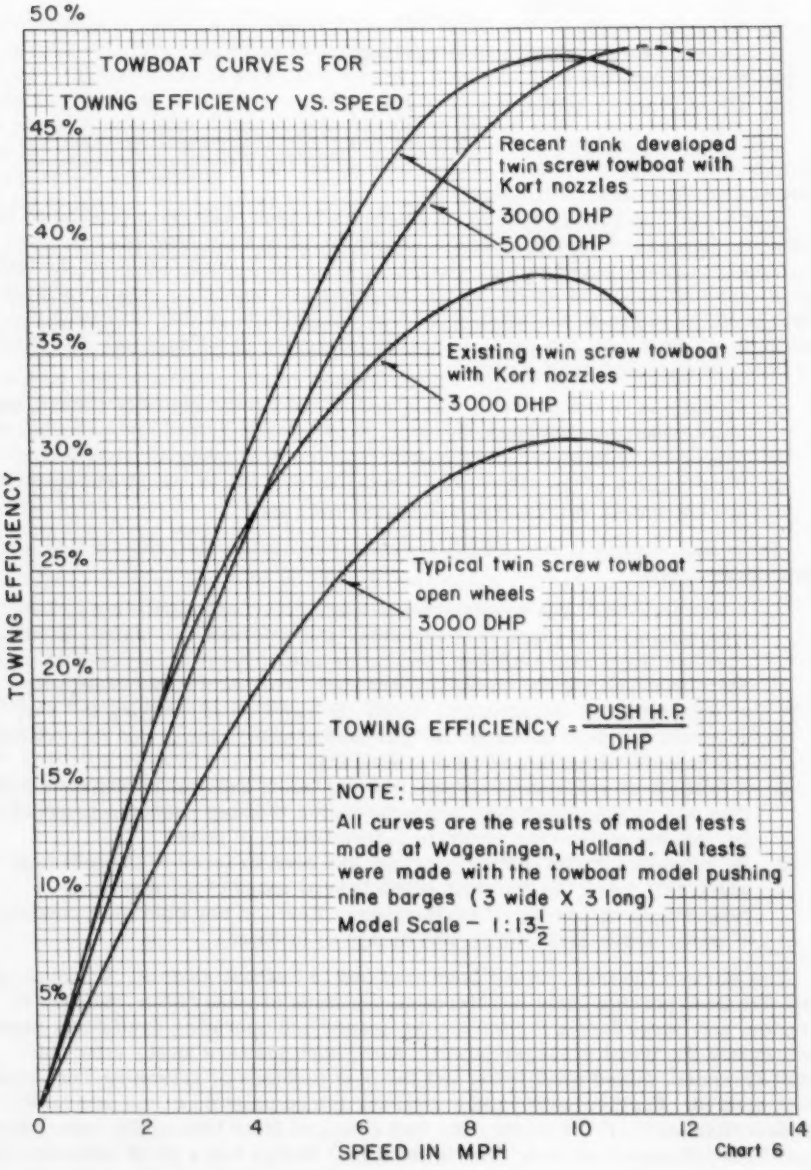
TOWBOAT CURVES FOR
TOWING EFFICIENCY VS. SPEED
Recent tank developed twin screw towboat with Kort nozzles
3000 DHP
5000 DHP
Existing twin screw towboat with Kort nozzles
3000 DHP
Typical twin screw towboat open wheels
3000 DHP
TOWING EFFICIENCY = PUSH H.P. / DHP
NOTE:
All curves are the results of model tests made at Wageningen, Holland. All tests were made with the towboat model pushing nine barges (3 wide X 3 long)
Model Scale – 1:13½
TOWING EFFICIENCY
50%
45%
40%
35%
30%
25%
20%
15%
10%
5%
0
2
4
6
8
10
12
14
SPEED IN MPH

Chart 6

6. Stern form variations.
7. Bow form variations.
8. Towboat length.
9. Towboat width and propeller spacing.
10. Kort Nozzle form.
11. Depth of water effect.

Each of these had to be evaluated for their effect on propulsion over the entire speed range of modern towing, as well as their influence on backing power and ahead and astern steering.

The boats that have been built on the basis of tank tests have confirmed the value of the method beyond question. However, it has not been possible, because of the great power of these boats, to get even a crude check on the tank predictions of push and speed. Also, the boats are operating with tows larger than could be tested in the existing tank at Wageningen so that operating results do not help. All the usual difficulties in trying to conduct full size trials of river fleets could apply as well:

1. Variations in water depth, width, and current over any trial course, and between successive runs.
2. Inadequate measurement of power.
3. Poor control of the effects of steering so that power, RPM and speed are constantly varying and difficult to average.

From this unsatisfactory situation have come certain decisions which are now taking effect:

1. A new tank has been constructed at Wageningen especially for river type testing. This tank is much wider and adjustable in width. It has a variable water depth as well, so that an actual river cross section can be reproduced. This tank will yield model test data close enough to actual operating conditions to justify the expense of carrying out very accurate full size trials for the purpose of determining any corrections that may be necessary to tank predictions.
2. A towboat of much smaller size is being built, and will be tested in full size and model. Complete instrumentation is being purchased, including torsion meters for accurate determination of power.
3. An extensive model testing program has been ordered in the new tank which will allow the accurate prediction of speed of all sizes, arrangements and drafts of modern barge fleets in any operating condition of water depth, boat power, and design of towboat.

The work in the tank has not been confined to towboat studies. An equal effort has been put into tests to determine the value of integration, semi-integration, and barge forms for the higher speeds and operating drafts that must accompany the trend to larger power. Also, such matters as the effect of draft on speed, mixed draft tows, and the best methods of arranging tows have been studied. Further work is now awaiting the completion of the new tank, as described earlier. However, one fact emerged from testing the same barge fleet with different towboats. The towboat hull design has a great influence on the push required for a given speed, just as Chart 3 shows the fleet to influence the towboat. For this reason, if one wishes to compare large fleet tests either in model or full scale, it is necessary to use identical towboats. One standard towboat model is now used for all barge tests.

## CONCLUSION

It is a logical question to ask why all this effort and expense when towboats and barges have been built and improved over the years without all this fancy business. First, there is no question in the minds of some pretty hard-headed people who operate and live from river equipment that this work pays off. That is not a scientific answer, however, so a typical case was calculated to show the value of a 10% improvement in efficiency under modern economic and operating conditions.

Two examples were chosen to predict what increase in initial investment could be made and repaid in five years from the increased profits, after taxes, resulting from a 10% increase in efficiency. The same basic operation was chosen as was used for the charts. Calculations were developed for a 4000 HP towboat with fleets of 15 barges and 30 barges.

For the boat and 15 barge fleet, an increase in initial investment of 11-1/2% can be returned in 5 years from the increased profits. With the same boat and 30 barges, the increased profits will return a 32% increase in initial investment in 5 years.

If the gains in Chart 6, which can be achieved at costs well under the allowable cost in the examples, are multiplied by the number of boats that can benefit from one test, and by the years of operation of each boat, a rather astronomical saving can be shown. The real point is that the technical situation in the design of river equipment has reached the area of certainty that has applied in the design of seagoing marine equipment for a long time. Only by the use of more science and less guesswork can boats and barges be improved, as well as operations, since only scientific information makes it possible to pre-select the most efficient methods with accuracy.

While scientific accuracy has not yet been completely demonstrated, enough has been learned of the value of these new and far more accurate methods of model testing to have no doubt but that predictions of performance of any river tow under any operating conditions within say 2% in speed are possible. The cost of such a prediction is low enough to come under the heading of miscellaneous in anyone's operating budget, and one cannot imagine a cheaper form of insurance for the purchaser and planner of river equipment.

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# HYDRAULIC DESIGN OF COLUMBIA RIVER BASIN NAVIGATION LOCKS

G. C. Richardson,[1] A.M. ASCE, and M. J. Webster,[2] M. ASCE
(Proc. Paper 1773)

## SYNOPSIS

Some of the highest lift navigation locks in the world are located on Columbia River Basin Projects. Locks at McNary and The Dalles have maximum lifts of 92 and 90.5 ft., respectively. These will be exceeded for a brief period by the new Wilson lock on the Tennessee River which will have a lift of 101 ft. The Ice Harbor lock on the Snake River, the main tributary of the Columbia River will have a maximum lift of 103 ft. when completed in 1961. This in turn will be exceeded by the John Day Lock, scheduled for completion in 1966, which will have a maximum lift of 113 ft. Many improvements which have been recently developed are incorporated in the design of the hydraulic systems for these locks. As a result of the increased lock lifts, the rapidly expanding volume of traffic and the demands of the navigation interests, the designers have been forced to develop many of these improvements, frequently with the aid of hydraulic model studies. The tremendous improvements in the filling systems are reflected by the almost complete lack of turbulence in the lock chambers of McNary and The Dalles locks as compared to the very turbulent condition in the Bonneville lock. The model studies indicate that even less turbulence will occur in the Ice Harbor lock.

## INTRODUCTION

The Columbia River has a project depth of 35 ft. from its mouth to the mouth of the Willamette River near Portland, Oregon, and 30 ft. to Vancouver,

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1773 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 4, September, 1958.

1. Chf., Hydr. Design Section, U. S. Army Engr. Dist., Walla Walla, Corps of Engrs., Walla Walla, Wash.
2. Chf., Hydr. Design and Lab. Section, U. S. Army Engr. Dist., Portland, Corps of Engrs., Portland, Ore.

Washington. Project depth from Vancouver to Bonneville is 27 ft., however, the channel has not been maintained to this depth. The depth of the pool above Bonneville Dam to The Dalles is 30 ft. or over. Although the navigation lock at Bonneville was designed and built to accommodate ocean-going vessels, traffic upstream from Vancouver has been limited to barge tows.

This paper is limited to a brief description of the hydraulic systems of the navigation locks at Bonneville, McNary, The Dalles, and Ice Harbor, to a comparison of the approaches to the Bonneville, McNary and The Dalles locks, and to a few problems encountered during operation. Bonneville lock was placed in operation in 1938; McNary in 1953, and The Dalles in 1957. Construction of Ice Harbor lock is scheduled for 1959 and 1960 and will be ready for river traffic in 1961. Locations of these projects are shown in Fig. 1. Data on existing and proposed locks in the Columbia River Basin are given in Table A.

## Bonneville Navigation Lock

Bonneville Dam is situated approximately 146 miles upstream from the mouth of the Columbia River. The navigation lock, on the Oregon side, is 76 ft. wide, has a usable length of 500 ft. and has a depth of 24 ft. over the lower sill at normal low water. Normal lift of the lock is about 58 ft., however, the lift can vary from 66.5 ft. at low water to 30 ft. at extreme high water.

The hydraulic system (see Plan, Fig. 2) is similar in design to the first Keokuk lock on the Mississippi River and to the locks in the Panama Canal; the lock chamber fills and empties through circular openings in the floor of the chamber. Water enters the lock from the powerhouse forebay through two intakes in the river side of the north or river wall. The two intake culverts, each equipped with a 7.0- by 11.5-ft. tainter valve, lead into a 14-ft.-diam. longitudinal central culvert beneath the floor of the lock with branches leading to 41 ports, each 4 ft. in diameter. The total area of the lock chamber ports is 515 sq. ft.; ratio of port area to culvert area at the filling valves is 3.2 to 1. The lock empties through the same culvert system leading to two 7.0- by 11.5-ft. tainter valves near the downstream end of the lock. The water discharges into the lower pool through five 6.0- by 7.0-ft. vertical ports located immdiately downstream from the miter gates. The lock chamber is equipped with three floating mooring bits in each wall.

There is extreme turbulence in the lock chamber during filling with normal operation of the filling valves (see Fig. 3). The valves are of the conventional tainter type with skin plates facing upstream and were designed to open in 2.3 min. Considerable air enters the lock through vortexes at the intakes and through the valve shafts. To reduce turbulence in the lock chamber when locking all types of traffic, the filling valves are opened to the 1/4 position and the chamber filled to about Elev. 30; the valves are then opened to the 1/2 position and the chamber filled to about Elev. 40 after which time the valves are fully opened. (Normal tailwater is Elev. 14.) This increases the filling time for the normal lift of 58 ft. from 11.5 min. to about 15 min.

The emptying valves, also conventional tainter type, are opened to full open position in 2.3 min. Flow from the large vertical discharge ports shoots several feet above tailwater causing a reduction in the effective head. Emptying time is about 13.3 min. for normal operation with a head of 58 ft., approximately 2 min. longer than the filling time. The violent action of the water

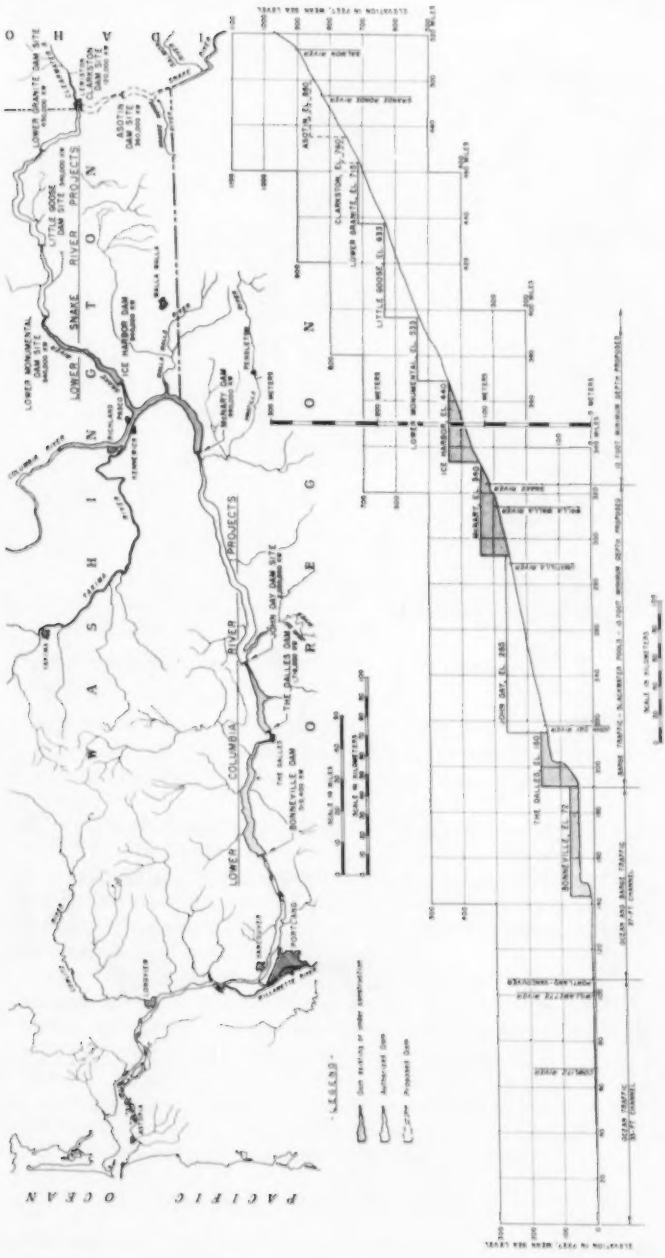

COLUMBIA RIVER PROJECTS
SNAKE RIVER PROJECTS
LOWER GRANITE DAM SITE
LITTLE GOOSE DAM SITE
LOWER MONUMENTAL DAM SITE
ICE HARBOR DAM
MCNARY DAM
JOHN DAY DAM SITE
THE DALLES DAM
BONNEVILLE DAM
PORTLAND
PACIFIC OCEAN
LOWER GRANITE EL. 733
LITTLE GOOSE EL. 633
LOWER MONUMENTAL EL. 537
ICE HARBOR EL. 440
MCNARY EL. 340
JOHN DAY EL. 265
THE DALLES EL. 160
BONNEVILLE EL. 72
OCEAN AND BARGE TRAFFIC
BARGE TRAFFIC

Fig. 1

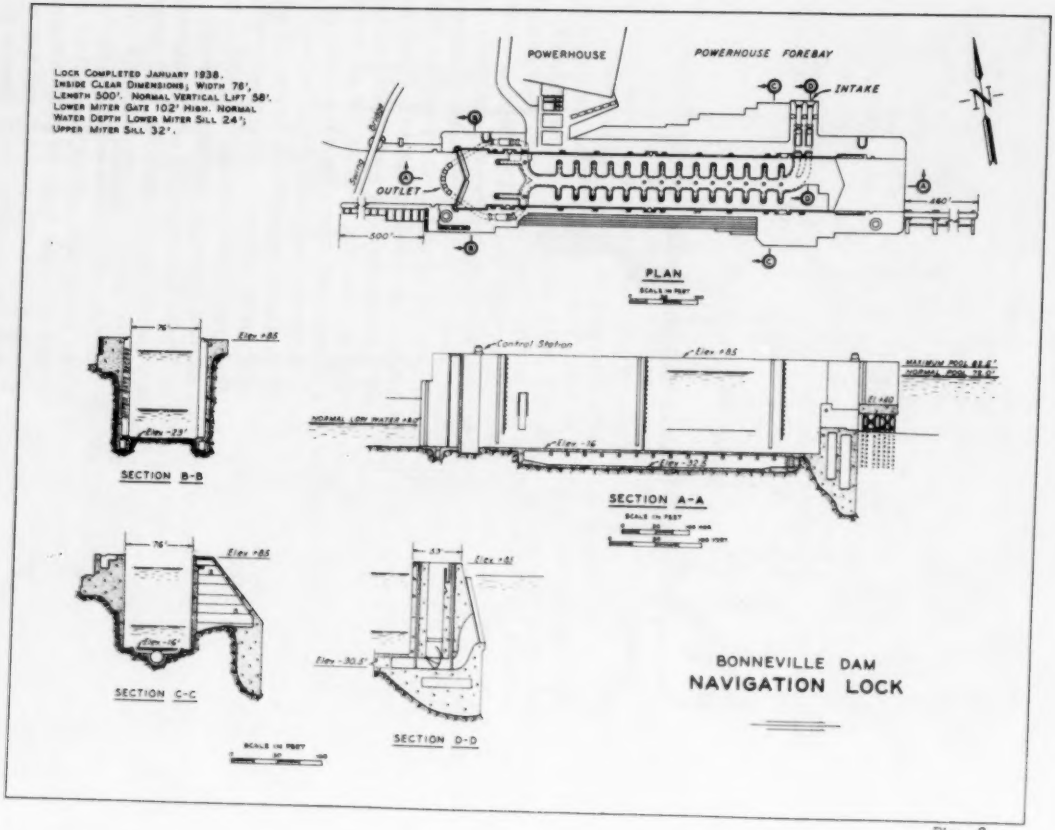

LOCK COMPLETED JANUARY 1938.
INSIDE CLEAR DIMENSIONS; WIDTH 76',
LENGTH 500'. NORMAL VERTICAL LIFT 58'.
LOWER MITER GATE 102' HIGH. NORMAL
WATER DEPTH LOWER MITER SILL 24';
UPPER MITER SILL 32'.
POWERHOUSE
POWERHOUSE FOREBAY
INTAKE
OUTLET
500'
460'
PLAN
SECTION B-B
Elev +85
Elev -29'
Control Station
Elev +85
NORMAL LOW WATER +82'
Elev -76
Elev -32.5
SECTION A-A
SECTION C-C
Elev +85
Elev -16'
Elev +85
Elev -30.5'
SECTION D-D
BONNEVILLE DAM
NAVIGATION LOCK

Fig. 2

Fig. 3. Turbulence in Conneville Lock During Filling

during emptying washes sand and gravel against the downstream side of the miter gates which deposit on the gate seats when the miter gates are opened. This has caused considerable damage to the gate seals resulting in high maintenance costs.

Flow conditions in the approaches to the lock are shown in Fig. 4 with a flow of 185,000 c.f.s. In the lower approach, flow from the spillway is deflected from the right bank across the navigation channel to the left bank. Upstream bound tows encounter considerable difficulty in navigating through high velocity flow in the narrow section of the river that extends from 1/4 to 1 mile downstream from the lower lock approach.

The narrow passage at the upstream end of Bradford Island causes considerable turbulence at the upstream approach. Flow into the powerhouse, especially during low flows, has a tendency to pull tows away from the lock entrance. A longer guard wall would improve conditions at the upstream approach. However, extending this wall would be extremely expensive due to the height of wall required. Work currently under way to remove approximately 26,000 cu.yds. of rock from Eagle Creek Point on the left bank will improve conditions for navigation in the upstream approach.

## The Dalles Navitation Lock

The Dalles Lock and Dam Project is located about 3 miles upstream from the city of The Dalles, Oregon, and about 192 miles upstream from the mouth

Fig. 4. Bonneville Dam

of the Columbia River. The navigation lock, located on the Washington side, is 86 ft. wide and 675 ft. long in the clear, the standard size adopted for Columbia River Basin locks. Minimum depth over the sills is 15 ft. The average and maximum lifts are 87.5 and 90.5 ft., respectively.

The hydraulic system for the lock is shown in Fig. 5. It was necessary to vary the design somewhat from the conventional hydraulic system because of the location of the lock in practically solid basalt rock. The two major variations were the location of the intakes at the bottom of a pit riverward of the lock entrance and placing the longitudinal culverts in the floor of the lock chamber. The lock walls, with a minimum thickness of 4 ft., were anchored to vertical faces excavated in rock.

Water enters the lock through a single bellmouth entrance in each main culvert. Location of the intakes at the bottom of the deep pit provides ample submergence to prevent air from entering the intakes through the formation of vortexes. A trash rack structure upstream from the pit prevents trash from entering the approach to the intakes. Flow into the lock chamber is controlled with reversed tainter valves just downstream from the intakes in each of the two 12-ft. wide by 14-ft.-high main culverts. Flow enters the lock chamber through seven lateral culverts connected to each main culvert. The lateral culverts in the floor of the lock are grouped in the middle third of the chamber. The lateral culverts are 5 ft. 6 in. high throughout but vary in width from 5 ft. 6 in. at the entrance to 2 ft. at the downstream end to provide more uniform flow from the lateral. Each lateral has six ports in each side wall. All ports are 16 in. wide but vary in height to give a uniform flow into the chamber during filling; ports in the four upstream laterals are 4 ft. high, those in the six center laterals are 3 ft. 1 in. high, and those in the four downstream laterals are 1 ft. 9 in. high. The sizes of the ports were determined by model study. Total area of the ports is 663.8 sq. ft. giving a ratio of port area to main culvert area of 1.98 to 1.

Although placing the main culverts in the floor of the lock chamber saved considerable rock excavation and provided sufficient submergence to eliminate negative pressures at the filling valves, it required a double bend at the entrances to the lateral culverts. As shown in Fig. 6, these entrances extend the full width of the roof of the main culverts. There is very little turbulence in the lock chamber during filling (see Fig. 7). The lock chamber is equipped with four floating mooring bits in each wall.

Flow from the lock during emptying is controlled by reversed tainter valves in the main culverts and discharges through six lateral culverts located just downstream from the miter gate (see Fig. 8). There are six ports in each side wall of each discharge lateral. The ports are 4 ft. 8 in. high and vary from 1 ft. 7 in. to 2 ft. 3 in. wide and have a total area of 653.5 sq. ft.; ratio of port area to main culvert area is 1.94 to 1.

The Dalles lock is the first on the Columbia River to use a submergible tainter gate at the upstream end of the lock. It was designed as a service gate only and cannot be used to augment filling the lock. The gate at the downstream end of the lock is a conventional miter gate.

The filling valves open at a uniform rate in 4.2 min. Observed filling time for a lift of 82 ft. was 10.7 min. The emptying valves open in 2.1 min. and lock emptying time for an 82-ft. head is 11.5 min.

All phases of the design of the hydraulic system were studied in a 1:25-scale model at the Bonneville Hydraulic Laboratory. Several revisions were made during the model studies especially in the size and spacing of the ports

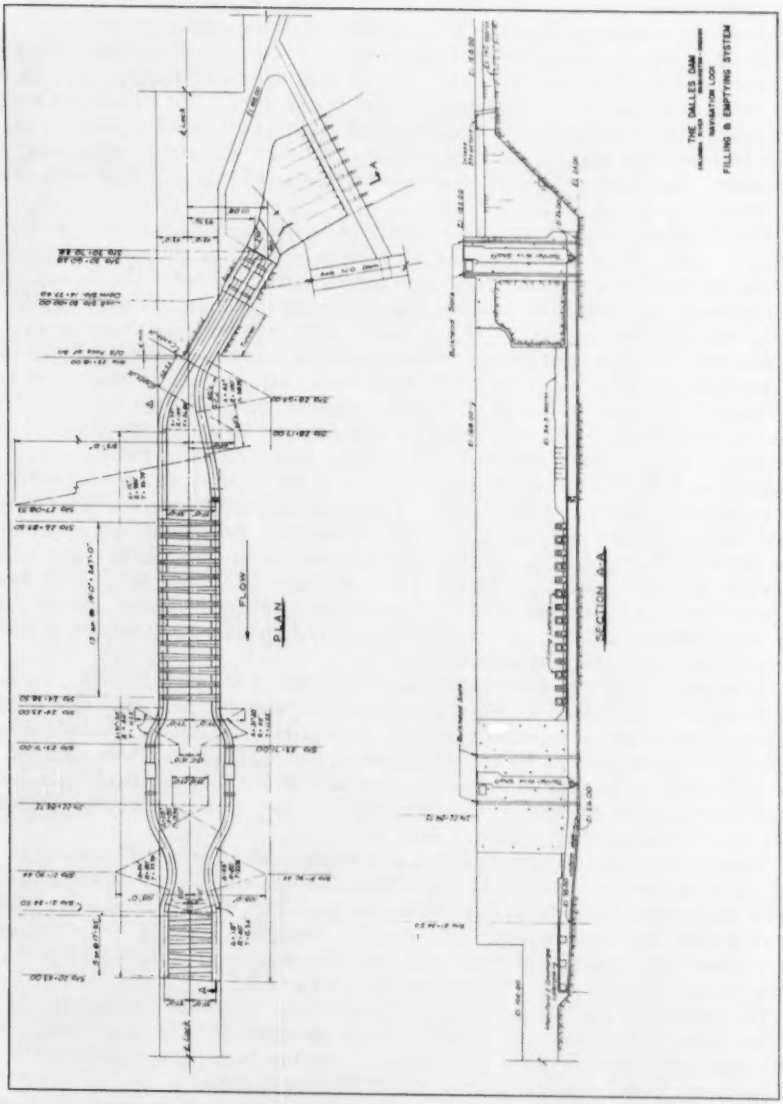

THE DALLES DAM
NAVIGATION LOCK
FILLING & EMPTYING SYSTEM
PLAN
FLOW
SECTION A-A

Fig. 5

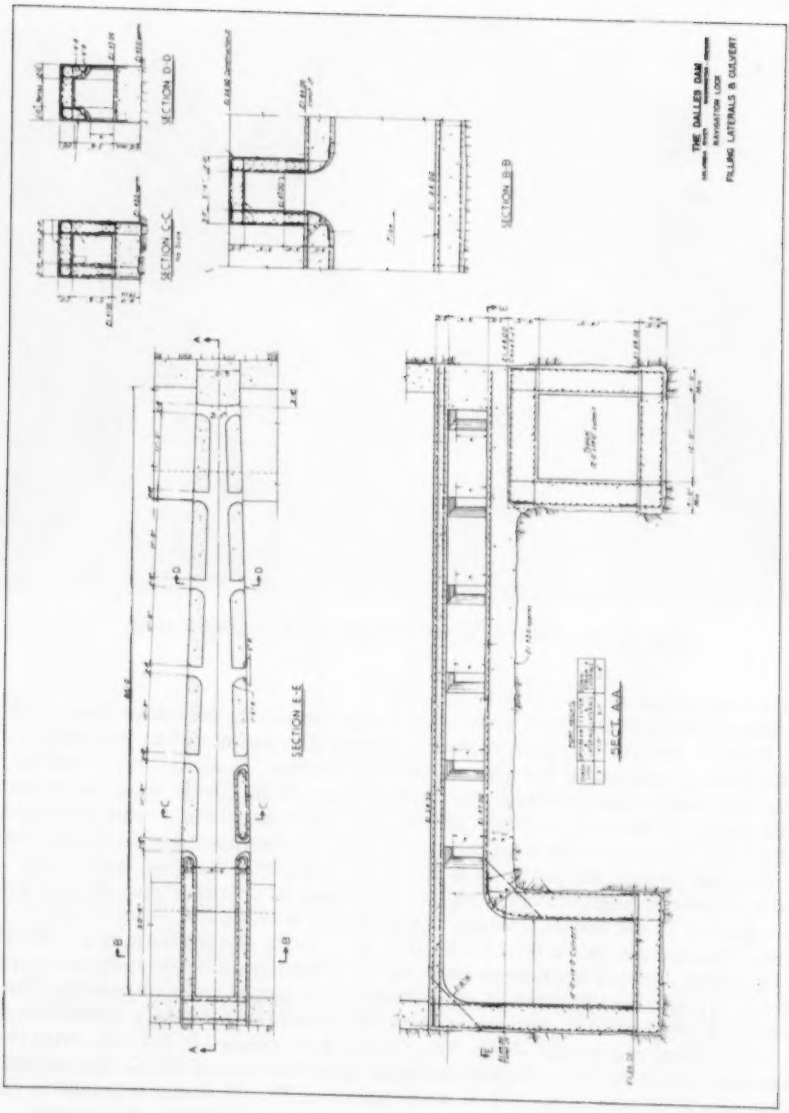

THE DALLES DAM
NAVIGATION LOCK
FILLING LATERALS & CULVERT
SECTION D-D
SECTION C-C
SECTION B-B
SECTION E-E
SECT A-A

FIG. 6

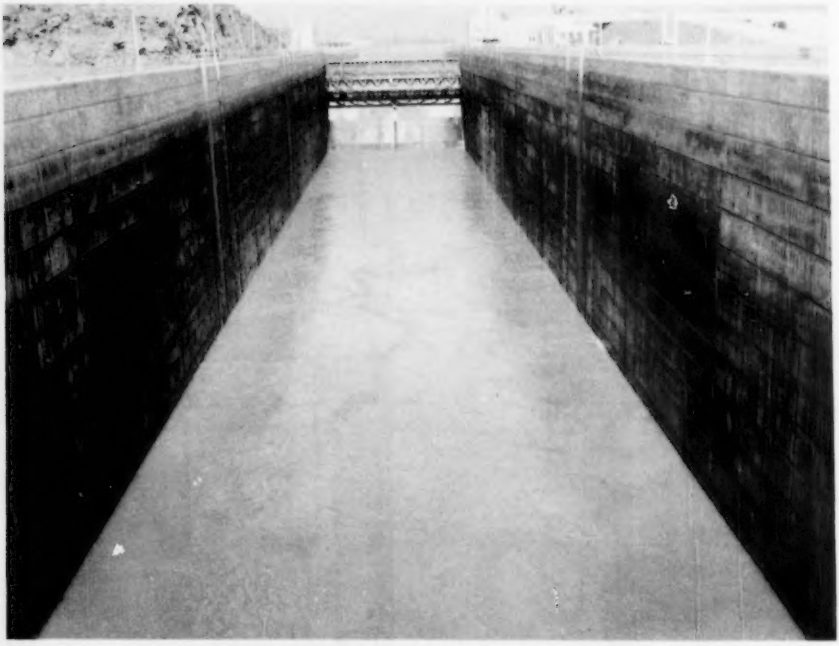


Fig. 7. Turbulence in the Dalles Lock During Filling

in the lock chamber lateral culverts.

The model indicated that vortical action occurred at the intake ports with the original design. This action was eliminated in the model by simulating additional excavation in the approach to the intakes. Vortical action has not been observed in the prototype for lifts to about 85 ft. Decreasing the height of the ports in the lock chamber lateral culverts toward the downstream end of the lock resulted in less turbulence in the lock chamber during filling. In the original design, all ports were 1 ft. 4 in. wide by 2 ft. 10 in. high. The model indicated better distribution from the lateral culverts with ports 1 ft. 4 in. by 4 ft. in the upstream four laterals, 1 ft. 4 in. by 3 ft. 1 in. in the center six laterals and 1 ft. 4 in. by 1 ft. 9 in. in the downstream four laterals.

Pressure observations were made in the model at all critical locations in the culvert system. In general only positive pressures were observed. The lowest pressure observed in the roof of the culvert immediately downstream from the filling valve was +4 ft. The valves were opened in 4.2 min. with the maximum lift of 90.5 ft. Pressures were observed during filling operations at several locations in the 3 ft. 6 in. radius (see Fig. 6) in the entrance to one of the lock chamber lateral culverts. There was no indication of negative pressure in the double bend to the lateral culverts.

Shortly after the prototype lock was placed in operation in March 1957, a noise having the indication of cavitation was noted in the wall culverts. The noise lasted for a period of 2 to 3 min. after the valves were opened. When the lock was unwatered in October 1957, a thorough inspection was made of

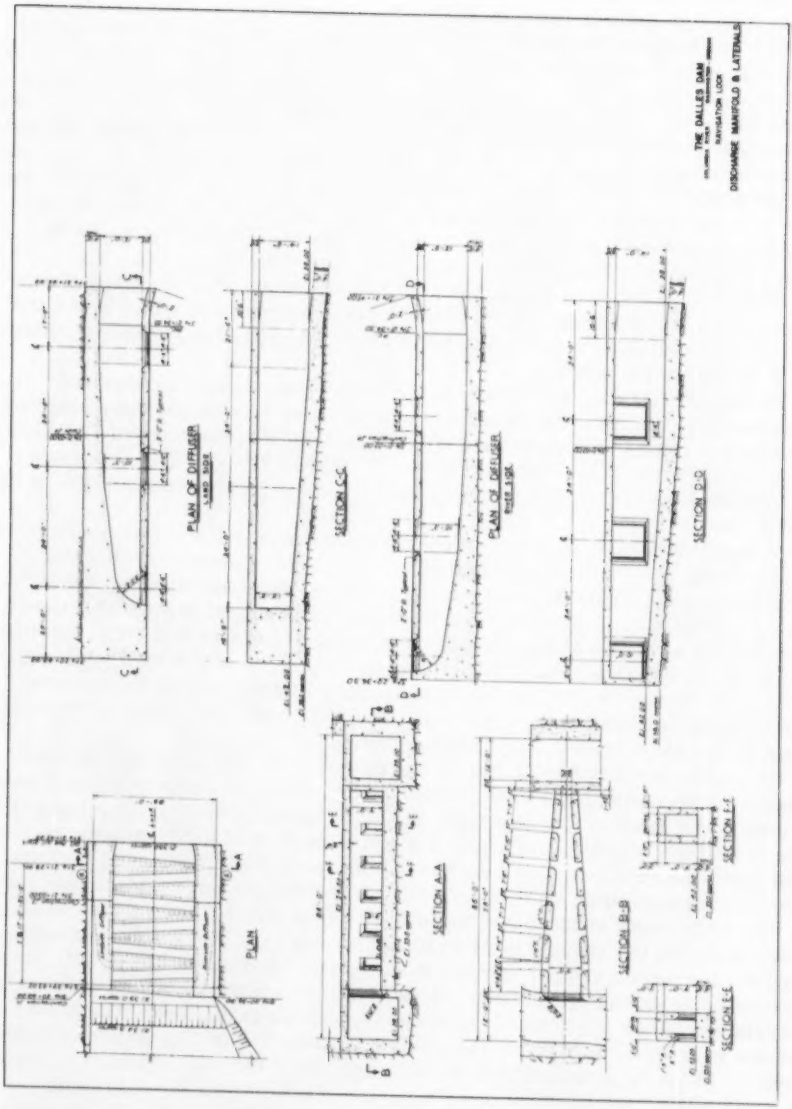

THE DALLES DAM
NAVIGATION LOCK
DISCHARGE MANIFOLD & LATERALS
PLAN OF DIFFUSER
SECTION C-C
PLAN OF DIFFUSER
SECTION D-D
PLAN
SECTION A-A
SECTION B-B
SECTION E-E
SECTION F-F

Fig. 8

the wall culverts and entrances to the lateral culverts. No damage was observed in the wall culverts, however, some erosion of the concrete was noted in the 3 ft. 6 in. radius curve (see Fig. 6) at the entrance to the lateral culverts. The erosion started in surface voids left in the concrete; absorptive liners were not used in the forms. Originally these voids varied from 1/8 to 3/8 in. deep, 1/2 to 1 in. wide and 3/4 to 1-1/2 in. long. The erosion of the voids varied from none in some locations to considerable in others. It is believed that localized cavitation at the voids is the cause of the noises observed on the lock walls during filling. Although damage to the concrete is slight at the present time, it will continue with operation of the lock and eventually the concrete entrances to the lateral culverts will have to be replaced. Absorptive form liners would have eliminated the surface voids and their use is highly recommended at similar locations in future locks.

Additional tests were made in the 1:25-scale model after the damage was observed in the prototype. Several more piezometers were added at the entrance to the lateral culvert. The tests showed no negative pressures at any point in the lateral entrance.

Hawser force tests were made in the model with various numbers of standard size barges. The maximum hawser force measured was a longitudinal force of 8 tons with eight barges having a total displacement of 11,040 tons when the lock was filled with the maximum head of 90.5 ft. The maximum transverse force, also with 8 barges, was 4 tons at the downstream end of the barge tow.

In December 1957, hawser force tests were made in the prototype locks at McNary and The Dalles.(1) Longitudinal forces were measured on a tow furnished by the Tidewater-Shaver Barge Lines, Portland, Oregon, having a total displacement of 2780 tons. A maximum longitudinal force of 3.2 tons was measured at The Dalles during filling with a lift of 84.6 ft. The maximum force during emptying was 1.1 ton. A maximum force of 2.8 tons was measured in the model in a filling test simulating prototype conditions for very satisfactory correlation between model and prototype. A hawser test was not made in the model for an emptying operation.

The approaches to The Dalles navigation lock provide very satisfactory conditions for navigation. The general alignment of the guide and guard walls are shown in the aerial photograph, Fig. 9. The concrete gravity guide walls are in straight alignment with and are a continuation of the right lock wall. The upstream guide wall extends 686 ft. upstream from the upper lock gate. The downstream guide wall extends 1020 ft. downstream from the miter gate.

The alignments of the guard walls were based on studies in a 1:80-scale river model at the Bonneville Hydraulic Laboratory. The upstream guard wall is of concrete gravity construction, extends approximately 1165 ft. upstream from the upper lock gate and is flared to a width of 307 ft. at the upstream end of the approach to provide easy access to the lock. The downstream guard wall is of rock fill construction and flares to a protected channel width of 250 ft. at a point 645 ft. downstream from the miter gate. The guard wall extends a total distance of 1320 ft. downstream from the miter gate.

The upper limit of navigation at The Dalles is 800,000 c.f.s. Model tests indicate maximum velocities of 6 to 7 f.p.s. at the upstream end of the upper guard wall and 6 to 8 f.p.s. at the downstream end of the lower guard wall for this discharge.

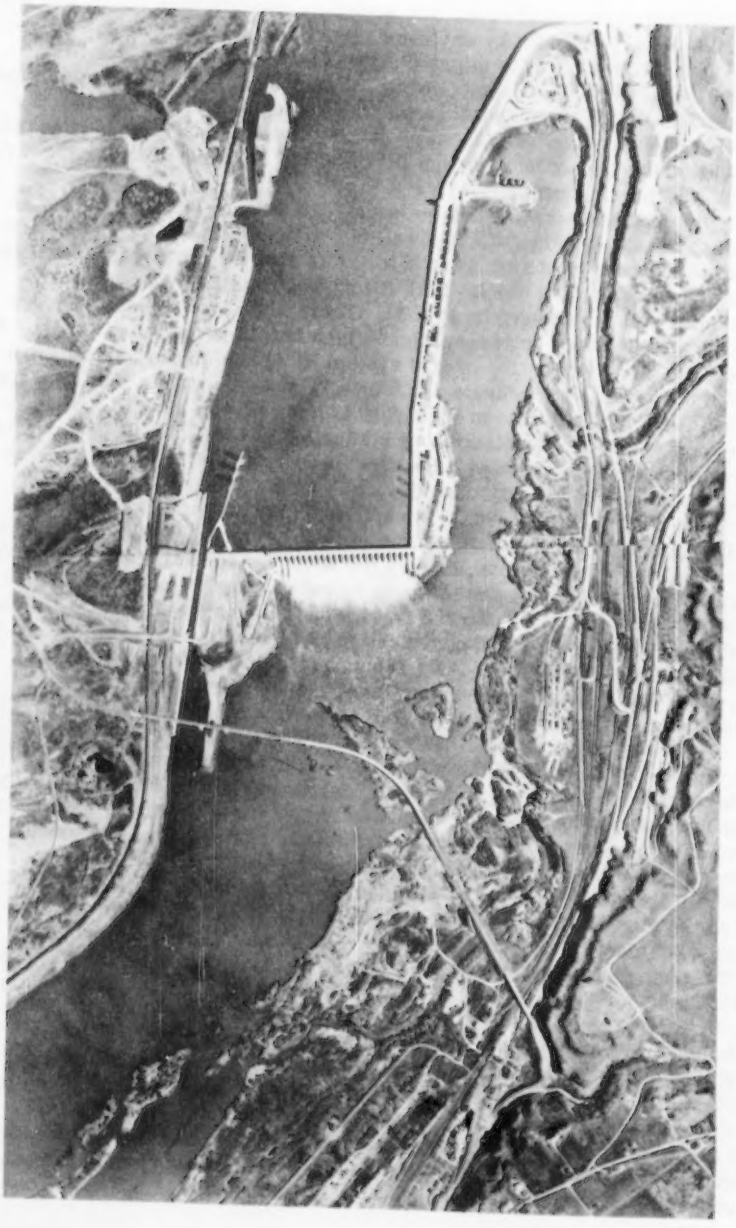


Fig. 9. The Dalles Dam

## McNary Navigation Lock

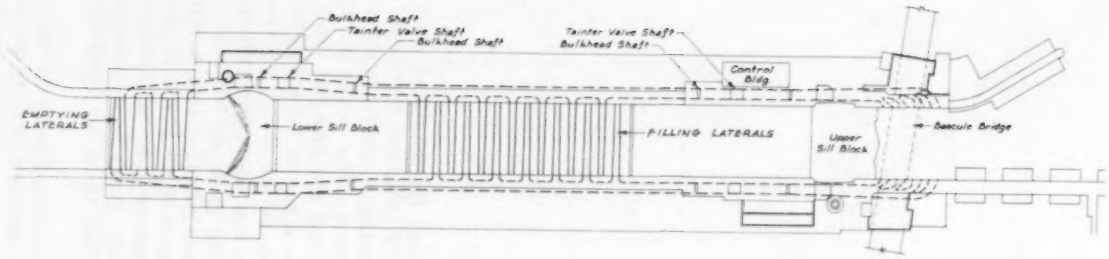
The first lock conforming to the now accepted standard size (86 by 675 ft.) for Columbia River Basin locks was constructed at McNary Dam, 292 miles above the river's mouth. It is situated at the foot of the former Umatilla rapids, 2.5 miles above Umatilla, Oregon. It provides slack-water navigation 36 miles upstream to Pasco and Kennewick, Washington and 10 miles up the Snake to Ice Harbor Dam. The lock, placed in operation in 1953 under normal pool elevation, has a maximum design lift of 92 ft., thus becoming the highest single-lift lock now in operation.

Filling the lock is accomplished by means of a bottom-lateral system, as shown schematically in Fig. 10. The wall culverts are 11 by 12 ft. in size, the intakes to which consist of four 11 ft. 6 in. by 14 ft. 9 in. openings in each wall immediately above the sill block. Both filling and emptying valves are reversed tainter valves, as seen on Fig. 11. There are seven lateral culverts extending from each wall culvert, each of which has 12 ports 1 ft. 2 in. by 2 ft. 6 in. (see Fig. 12). The entire lateral system is located in the central third of the lock chamber. The laterals are 5 by 5 ft. at the entrance, tapering uniformly by 2 by 5 ft. The ratio of total port area to culvert area is 1.86 to 1.

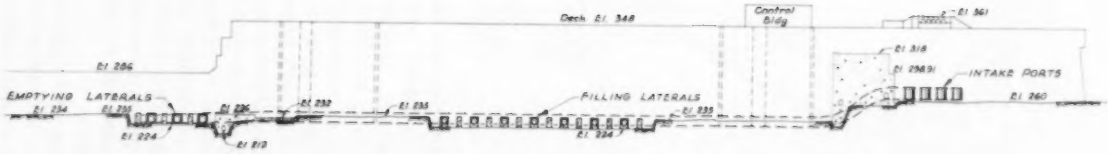
The emptying system consists of a continuation of the 11 by 12 ft. culverts into the downstream approach walls, where three laterals from each culvert extend into the floor of the approach channel. Each discharge lateral is tapered from 8 by 8 ft. to 2 by 8 ft. in section and includes 16 ports of constant height, 2 ft. 6 in., and of a width varying from 1 ft. 11 in. to 1 ft. 5 in.

When design was initiated on McNary the approved lock size was 86 by 500 ft.; initial design and model studies were based on these dimensions. When the decision was made to use the 675 ft. length, little time was left to complete model studies. For that reason construction had already started before all the model studies were performed. As a result one undesirable characteristic was revealed that could have been prevented had time permitted. This was the presence of low pressures on the roof of the culvert below the filling valves — pressures well down in the cavitation range with a valve opening time of 4 min. To alleviate the low pressures a nonuniform valve opening schedule was developed wherein the valve is opened rapidly 2 ft. then slowly until the lock has partially filled, then rapidly again, requiring 7.5 min. to become full open. In this way enough back pressure can be built up to enable the valve to pass through the most critical range — near 7 ft. — without seriously low pressures. Even so, pressures equivalent to -15 ft. were recorded in the model. This was assumed to be not dangerous in view of the stainless-clad plate liner below the valve — an assumption which proved to be true. The filling time as determined in the model was 16.1 min. for 92 ft. of head, 15.4 min. for 82 ft. and 14.1 for 63 ft. The time to empty was slightly faster, 15.7 min. for 92 ft., 15.2 min. for 82 ft. and 14.1 min. at 63 ft.

Turbulence in the lock chamber has been very low, due to the well distributed flow and the relatively slow filling time. With the lock chamber at its full capacity of eight barges the maximum hawser force, as measured in the model, was only 3.3 tons, and correspondingly less with small tows. Fig. 13 is a photograph of the lock chamber during a normal filling operation, taken immediately after the filling valves have become fully open — the condition producing maximum turbulence. One 12 in. air vent is open at each filling valve.




- PLAN -




- VERTICAL SECTION -

Fig. 10. McNary Lock

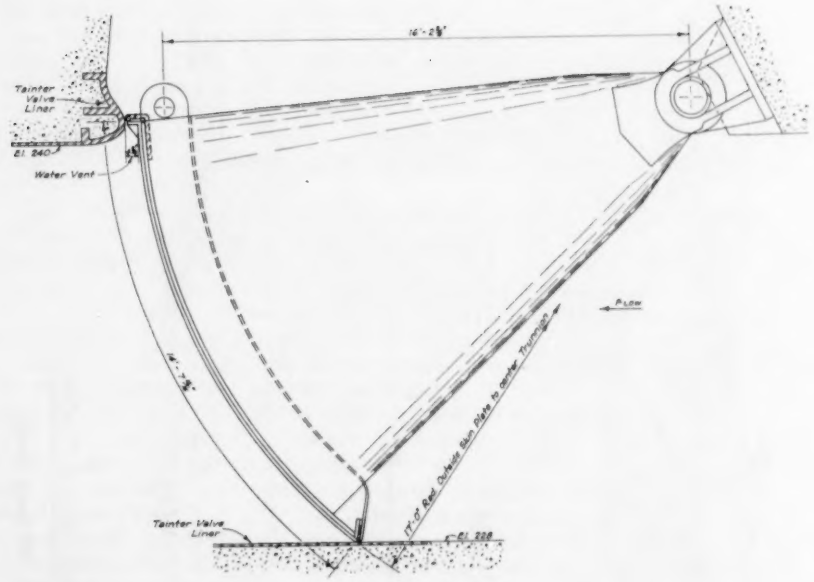
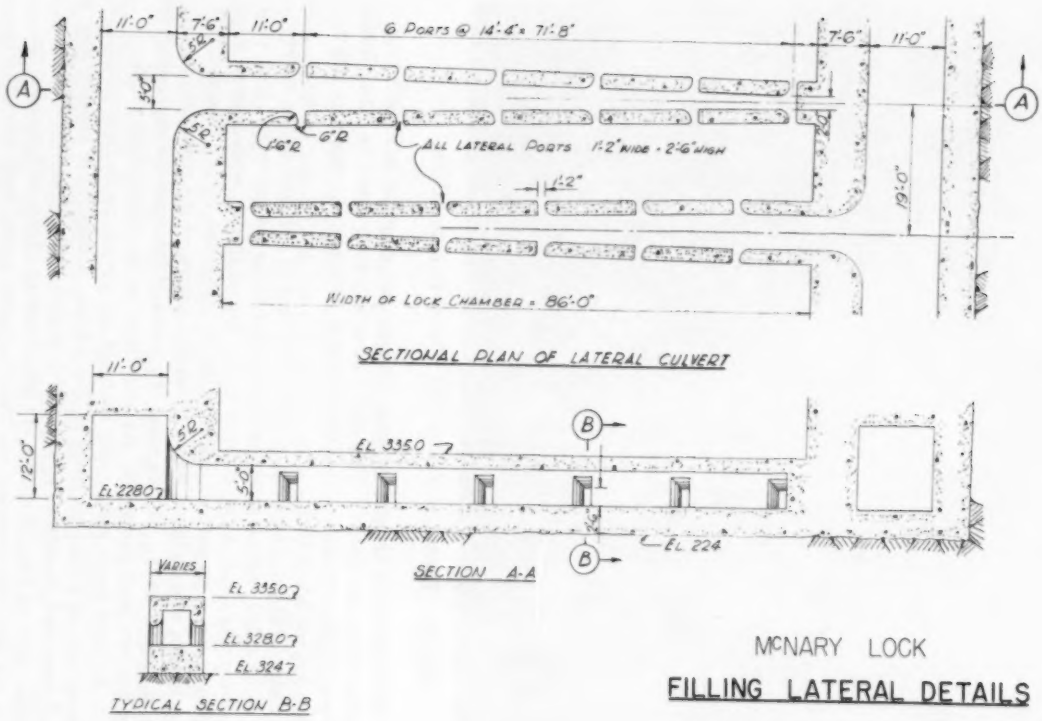



Fig. 11. McNary Lock Valve

Considerable attention was given to the proper shape for the filling valve in order that low downpull forces could be combined with minimum vibration. The resulting shape, illustrated on Fig. 11, incorporates an upstream convex skinplate with fully inclosed, streamlined trunnion arms. The valves at The Dalles have the same shape, as will those at Ice Harbor and John Day locks. Since McNary is the first of the high lift locks to use a bottom lateral system, much interest has been shown in its operation, which, on the whole, has been very good. When it was first put into service, some fear was expressed by observers that certain loud noises emanating from the valve and bulkhead shafts were indicative of damage to the valves. These noises, which appear to be acoustic phenomena associated with the formation of the hydraulic jump in the culverts during valve opening periods, have proved to be harmless, as no damage to valve, liner or machinery has appeared. It has been found that much of the sound can be reduced by opening one of six 12-in. air vents below each filling valve.

An extensive prototype testing program has been carried on by the Walla Walla District in conjunction with the Waterways Experiment Station and the Bonneville Hydraulic Laboratory. In October and November 1955(2) pressures in the filling and emptying culverts were measured by a system of 50 piezometers which had been installed at the time of construction. In general there was very good correlation between the model and prototype measurements. In addition filling and emptying curves were obtained, over-travel and under-travel observed and the effect of varying valve opening curves studied. It was




McNARY LOCK

## FILLING LATERAL DETAILS

Fig. 12

Fig. 13. Turbulence in McNary Lock During Filling

noted that the lock filled about one minute faster and emptied two and a half minutes faster than the model. This difference has been noted on other locks and is believed to be caused by the inability of a small scale model to reproduce friction losses accurately.

In November 1957 pressure cells were installed in the roof of the culvert near one filling valve, accelerometers were placed on the valve and lock wall and air-water probes set in the culvert roof. Air-mercury manometers were also attached to the piezometers adjacent to the pressure cells. The purpose of this installation was to learn more about the nature of the pounding noises coming from the culverts. At this time the mass of information obtained from these tests has not been analyzed. As mentioned in the discussion of The Dalles Dam, hawser force tests were made in the prototype in December 1957. With the same tow of 2780 tons used at The Dalles, a maximum longitudinal force of 1.7 tons was measured in the McNary lock with a head of 87.9 ft.

The aerial view in Fig. 14 shows the relationship of the lock and its approaches to the adjacent structures. The upstream guard wall extends for 1590 ft. from the upper lock gate and the downstream wall 1675 ft. from the lower gate. In addition a concrete moorage wall parallels the lower guard wall on the land side of the channel, which is 250 ft. wide at this point. Although extreme turbulence is formed over the discharge laterals during the emptying of the lock, conditions downstream where entering tows must wait, are quite acceptable.

Fig. 14. McNary Dam

Ice Harbor Navigation Lock

Ice Harbor Dam is located at the head of the McNary pool on the Snake River, about 10 miles from its mouth. First-stage construction, which includes the powerhouse, seven of the 10 spillway bays and the south abutment is well under way, and should be complete early in 1959. Construction of the navigation lock and the remainder of the north shore structures will commence as soon as the second-step cofferdam can be completed after the high water of 1959. The lock is scheduled to be put in operation late in 1961.

With McNary pool at its normal elevation of 340 and average flow conditions in the Snake maximum lifts up to 100 ft. can be expected annually. Were the McNary reservoir to be drawn down to its minimum operating level, Elev. 335, and nothing released from Ice Harbor, an absolute maximum of 105 ft. could occur. However, this would require a rather unlikely combination of circumstances, and for design purposes a head of 103 ft. has been used, exceeding both The Dalles and McNary.

Design of the lock is well advanced and a 1:25-scale model is being tested at the Bonneville Hydraulic Laboratory. Since the preliminary results have been very encouraging, it is anticipated that changes in the final design will be minor.

The plan dimensions of the lock are standard, 86 by 675 ft., but several unusual features warrant attention. First, the downstream lock gate will be a vertical lift gate, 86 by 89 ft., the top of which will seat against a concrete

box girder which forms the service bridge across the lock. The capability of such a gate being opened under slight head (up to about 3 ft.) permits the use of river discharge system, which is the second feature of special interest. Heretofore designers of Columbia River locks had been plagued by the difference in water surface between the river proper and the downstream lock approach — as much as 2 ft. in McNary. This condition necessitated the use of a lock discharge system in the lock approach since a conventional miter gate could not open against a head greater than 9 to 12 in. Thirdly, the filling system will be the split-lateral variation of a bottom lateral system. In this arrangement each of the two main culverts will have five lateral culverts in the lock floor (see Fig. 15). Instead, however, of the laterals interlacing as at McNary and The Dalles, one set of five will be located in the central third of the upstream half of the lock and the other five in the central third of the downstream half. In this way the magnitude of the longitudinal surge is reduced greatly, with resultant lower hawser forces. It is believed that Ice Harbor is the first high lift lock in this country to use the split lateral system in this fashion. Although the new Wilson lock will have a separate group of laterals from each of the two culverts, and in that sense has a split lateral system, the two groups are located adjacent to each other in the center of the lock chamber. The new Ohio River locks now under construction at Markland and Greenup, although similar to Ice Harbor in the separation of the lateral groups, will have appreciably lower lifts — 35 ft. at Markland and 32 ft. at Greenup.

The arrangement of culvert intakes is similar to that at McNary, having four ports, 8 ft. wide by 30 ft. high, in the channel side of each approach wall. Flow will be regulated by a reversed tainter gate, 12 by 14 ft, in each culvert. In order to reduce velocities at the lateral culvert intakes and to minimize excavation in the lock floor, each main culvert will be enlarged to 12 by 20 ft. opposite its group of five laterals and will be reduced again to 12 by 14 ft. at the emptying valves. Each lateral will be 8 ft. wide by 4 ft. high at its entrance and will step down in width at each of the six pairs of 1 ft. 3 in. by 2 ft. 6 in. ports, becoming 3 ft. wide at the end. The ratio of total port area to valve area is 1.11 to 1. For details of filling laterals see Fig. 16. Below each emptying valve, the conduit makes a horizontal 90 degree bend and extends into the river. The terminus of each conduit is flared from 12 by 14 ft. to 17.4 ft. to 19.4 ft. to reduce the exit loss and create a greater effective head.

Prior to the testing of the entire lock hydraulic system a model of a single lateral culvert was built at a scale of 1:16. It was realized that the split-lateral system was more sensitive to nonuniform flow distribution than the standard bottom laterals, which are somewhat self adjusting. Many combinations of flow and port arrangements were tried in this model. It was found that to get the optimum flow distribution would require a different design for each lateral. In the interests of simplicity a single design was selected to be common to all laterals. The differences in flow distribution between the selected design and various designs giving nearly uniform flow were felt to be negligible and indeed proved to be so when installed in the general model.

The preliminary model results have been gratifying. Even with the small port-to-culvert ratio of 1.11:1 the lock fills in 11.8 min. with a lift of 103 ft. with a constant speed valve time of 4 min. It empties in 13 min. with a valve time of 2.2 min. Experience at McNary and The Dalles indicates that the actual prototype filling and emptying may be faster. Pressures on the roof

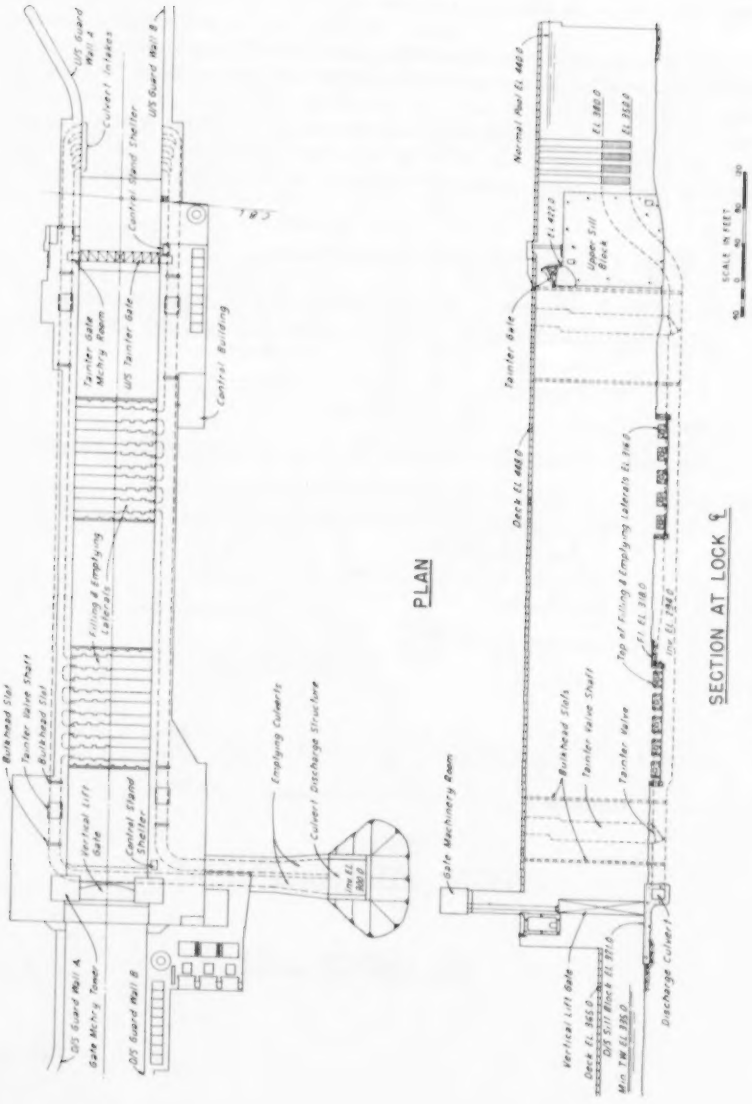
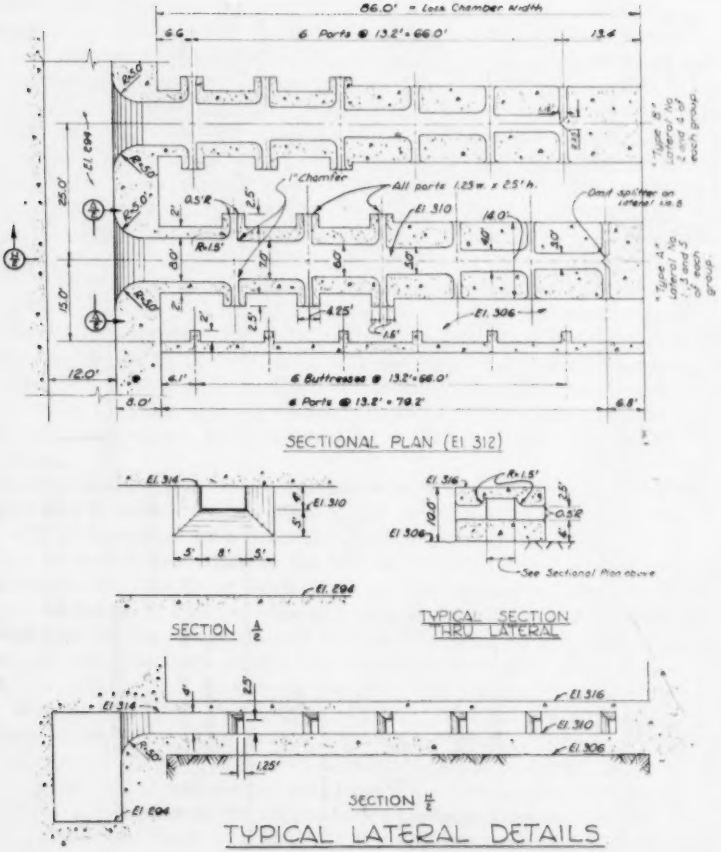



Fig. 15. Ice Harbor Lock




## ICE HARBOR LOCK

Fig. 16

below the filling valve are slightly positive when using both valves and but slightly negative under single valve operation with the 103 ft. lift. Hawser pull tests have just begun but they indicate that with an eight-barge tow the maximum force will be less than five tons.

The upstream and downstream guard walls will be relatively short since the general model of the Ice Harbor project revealed no adverse currents at either end of the lock. The upstream wall will extend 680 ft. from the face of the upper lock gate and the downstream wall 650 ft. from the lower gate. Since the lock empties directly into the river there will be no flow in the lower approach and hence no velocity. The guide walls will be of concrete.

## ACKNOWLEDGMENT

The authors wish to acknowledge the use of material from the Walla Walla and Portland Districts, U. S. Army, in the preparation of this paper.

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TABLE A

NAVIGATION AND POWER FACILITIES AT EXISTING AND PROPOSED DAMS
COLUMBIA - SNAKE RIVER SLACKWATER NAVIGATION SYSTEM

| Project | Mean River Discharge | Normal Pool Elev. | Lock Dimensions | | | Lock Gates (Lower) | | Power Facilities | | | | Status |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Width | Length | Max. Lift | Type | Ht. | Normal Max. | Head Min. | Installed Initial | Capacity Ultimate | |
| | c.f.s. | | ft. | ft. | ft. | | ft. | ft. | ft. | kw | kw | |
| Bonneville | 199,000 | 72 | 76 | 500 | 66.5 | Miter | 102 | 68 | 26 | 518,000 | 1,000,000(b) | (1) |
| The Dalles | 194,000 | 160 | 86 | 675 | 90.5 | Miter | 107.5 | 91 | 60 | 1,092,000 | 1,716,000 | (2) |
| John Day (a) | 188,000 | 265 | 86 | 675 | 113 | | 127 | 104 | 78 | 1,200,000 | 2,000,000 | (3) |
| McNary (c) | 185,000 | 340 | 86 | 675 | 92 | Miter | 106 | 92 | 62 | 980,000 | 1,400,000(b) | (1) |
| Ice Harbor | 48,500 | 440 | 86 | 675 | 103 | Vertical Lift | 89 | 102 | 77 | 270,000 | 540,000 | (2) |
| Lower Monumental(a) | 48,400 | 533 | 86 | 675 | 96 | | 112 | 96 | 75 | 270,000 | 540,000 | (4) |
| Little Goose (a) | 47,700 | 633 | 86 | 675 | 103 | | 119 | 103 | 85 | 270,000 | 540,000 | (4) |
| Lower Granite (a) | 47,600 | 715 | 86 | 675 | 85 | | 101 | 85 | 59 | 225,000 | 450,000 | (4) |
| Clarkston (a) | 31,100 | 760 | 86 | 675 | 48 | | 64 | 45 | 21 | 120,000 | 160,000 | (5) |
| Asotin (a) | 31,100 | 860 | 86 | 675 | 103 | | 119 | 100 | 87 | 360,000 | 450,000 | (5) |

NOTES: First four projects on Columbia River, remainder on Snake River.
(a) Data are preliminary. (b) Possible future increase considered. (c) John Day Dam will reduce head at McNary

Project Status:
(1) Initial development complete.
(2) Under construction.
(3) Authorized and preliminary design in progress.
(4) Authorized.
(5) Under study.

Paper 1781 September, 1958 WW 4

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

# THE BEAUHARNOIS CANAL LOCKS

Duncan McIntyre[1]
(Proc. Paper 1781)

## SYNOPSIS

In the Soulanges Section of the St. Lawrence river a power canal 15 miles long provides a deep water navigation channel from Lake St. Francis to within 10,000 feet of deep water in Lake St. Louis near Beauharnois, Quebec. Canalization of the remaining 10,000 feet is now being completed by the St. Lawrence Seaway Authority, present construction taking the form of two single separate locks with approaches and a short connecting channel. The fact that these facilities had to be fitted into a limited space, and that most of the required excavation had to be made in an extremely durable sandstone, posed a number of problems in structural and hydraulic design which were unique to the Soulanges Section of the St. Lawrence Seaway. This paper discusses the hydraulic features of Seaway structures designed for the Beauharnois site with emphasis on adaptations to site requirements.

Ship locks constructed by The St. Lawrence Seaway Authority near Beauharnois, Quebec, will complete the deep water navigation link between Lake St. Francis and Lake St. Louis through the Soulanges Section of the St. Lawrence river. In this section, as in much of the upstream reach to Lake Ontario, a channel excavation is being made as a power development enterprise under the stipulation that it provides also for navigation requirements. Excavation of the Beauharnois Power Canal, shown in Plate 1, was started in 1929 by a private company under an agreement with the Canadian government that 600 feet of width adjacent to the north canal bank would be excavated to the depth required for Seaway navigation. The power project was taken over by the Quebec Hydro Electric Commission in 1944 and the Commission is now completing channel excavation. Present construction by

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1. Sr. Asst. Hydr. Engr., The St. Lawrence Seaway Authority, Montreal, Quebec.

THE ST LAWRENCE SEAWAY AUTHORITY
THE SOULANGES SECTION OF THE ST LAWRENCE RIVER
LAKE ST. LOUIS
LAKE ST FRANCIS
ST. LAWRENCE RIVER
SOULANGES CANAL
POWER CANAL
BEAUHARNOIS
CEDARS
CASCADES POINT
POWER HOUSE
DAM
DE SALABERRY ISLAND
DAMS
BELLERIVE
VALLEYFIELD
COTEAU LANDING
DYKE
LOCK
LOCKS
UPPER LOCK
LOWER LOCK
NEW YORK - CENTRAL RAILROAD
OLD BEAUHARNOIS CANAL
CANADIAN PACIFIC RAILWAY
B.N.S.
PLAN 1

the Seaway Authority comprises the installation of locks with approaches between Lake St. Louis and the powerhouse head race, together with the provision of railroad and highway crossings.

Seaway locks and approaches had to be fitted into a limited space of approximately 10,000 feet between deep water in Lake St. Louis and a shoulder provided for entrance of the navigation channel near the lower end of the power canal. Plate 2 is an aerial view of the Beauharnois site indicating a main Provincial highway and a branch of the New York Central railroad for which Seaway channel crossings were required.

Early planning by engineers of the Canadian Department of Transport was based on the use of twin flight locks to be located adjacent to the Beauharnois powerhouse. On taking over responsibility for Seaway planning and construction in 1954, the Seaway Authority considered this scheme together with two others, one requiring a single highlift lock, the other comprising two single locks with a short connecting channel. Estimates indicated that the initial cost of flight locks would be considerably greater than that of either

PLATE 2

AERIAL VIEW OF BEAUHARNOIS SITE

a single high lift or single separate locks. However, plans for the latter two schemes required the future installation of second, or twin locks, so that, in their final development stages, there was not a great deal of cost difference between the three schemes.

Twin flight locks were finally discarded, chiefly because of their inherently poor hydraulic characteristics, and because they would not be adaptable to possible future Seaway needs. With reference to poor hydraulic characteristics, operating experience at the Welland Canal has shown that a troublesome surge is set up in flight locks caused by overtravel of water levels in lock chambers at the end of a lockage operation. This overtravel of water level at the end of a lockage operation may be noted in any lock using an extensive filling and emptying conduit system and is due to the inertia of water in the conduits. The amount of overtravel depends on conduit and lock chamber dimensions, model studies for Beauharnois locks indicating from 10 to 12 inches on the filling operation, with from 3 to 4 inches on emptying. When water level in the chamber has reached its limit of overtravel, flow in the conduits is reversed. If lock valves remain open and the lock miter gates are kept closed, there will be several such reversals with water level in the chamber fluctuating above and below the pool level outside. The amplitude of these flucutations decreases rapidly with time because of flow losses in the conduit system. In flight locks, where one chamber is emptied directly into another, a surge of considerable magnitude is developed by the overtravel phenomenon, particularly if the miter gates dividing the two chambers begin to open at the moment when water levels are first equallized. This surge travels the full length of both chambers causing an undesirable disturbance for a ship using the locks. It can be effectively damped out in the conduit system by keeping the miter gates closed for a few minutes after water levels are first equallized, but the lockage time is thereby increased.

Flight locks were not considered adaptable to possible future Seaway needs because twin chambers would have had to be installed initially in order that two-way traffic could be handled with reasonable facility. These twin chambers would occupy most of the space available for locks at the site so that future construction would be prohibitively expensive should it be deemed advisable to provide larger Seaway locks at a later date.

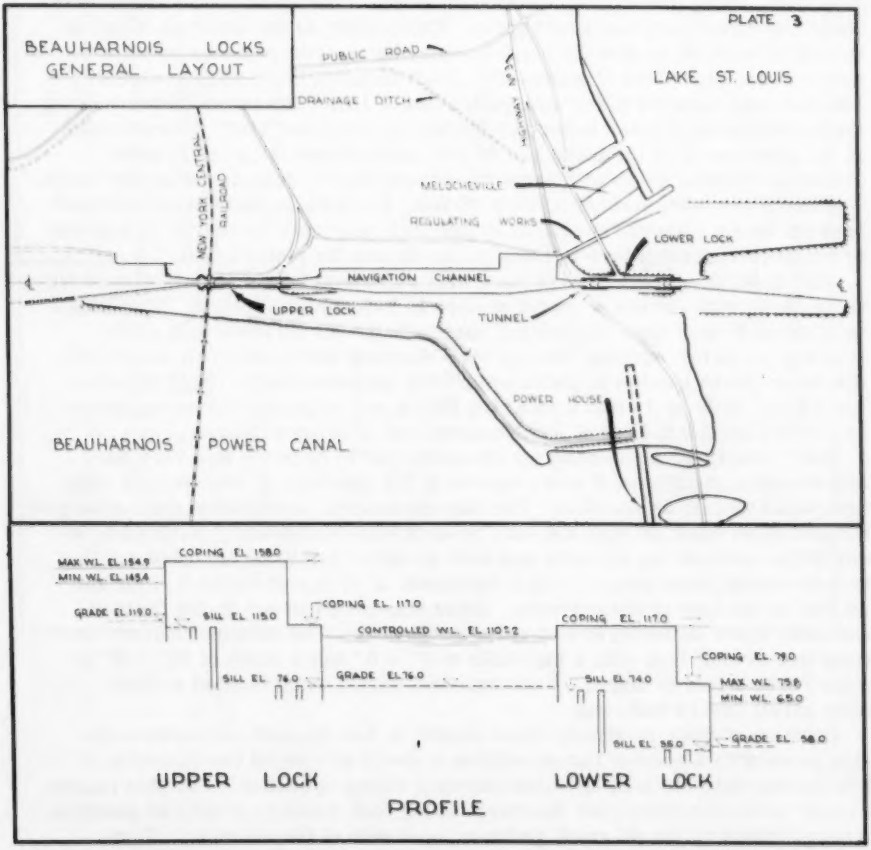
Had there been time available for the necessary model studies and design work, it is possible that a single high lift lock might have been found suitable for the Beauharnois site. The topography and excellent foundation conditions are well adapted to this type of structure. However, a lock of this type would have required the development of entirely new designs for the hydraulic system, miter gates, and much of the operating machinery. Very extensive and time consuming model studies would have been required for both structural and hydraulic features.

Another unfavorable attribute of the high lift lock was its potentially high consumption of water. A single high lift lock would use approximately twice as much water per filling and emptying cycle as would flight locks or two single separate locks. In terms of output at the power plant, this difference in water consumption would amount to approximately 4,000 K.W.H. of electrical energy per lockage cycle.

Although installation of the scheme comprising single separate locks with a short connecting channel presented some formidable problems, there were a number of decisive factors in its favour. Chief among these were the following:

1. The lift at each lock is not appreciably greater than the maximum lift at the upper, or Cote Ste. Catherine lock, at Montreal. The filling and emptying systems, operating machinery, and miter gates of the Beauharnois and Montreal locks could, therefore, be made similar, thus greatly simplifying the work of design, construction, and maintenance.
2. Provision is easily made for construction of second, or twin locks to be installed later to a revised design predicated on future Seaway requirements.
3. The scheme provides similar lockage facilities to those now being installed in other sections of the Seaway channel, thus assuring a smooth flow of traffic.

A plan view of the scheme selected for the Beauharnois site is shown in Plate 3. As indicated in this plan the downstream approach to the lower lock entends into Lake St. Louis, while the lock itself is placed just inshore. The



upper lock is located roughly 4,000 feet upstream just south of the New York Central Railroad line with its upstream approach extending into the power canal. A sector-type guard gate is provided in an upstream extension of the upper lock, with a swing span for the New York Central Railroad placed between the guard gate and the upstream miter gates of the lock. Regulating works for controlling water level in the channel between locks consist of a supply conduit bypassing the upper lock on its south side, and a side channel spillway with chute located north of the lower lock. A tunnel for Provincial Highway No. 3 passes under the chute and the upper end of the lower lock. The channel between locks has a vertical concrete face along its entire south wall and its bottom width initially will be 300 feet. Plans for future construction call for second, or twin locks to be installed on the north side of those indicated in Plate 3, together with a further widening of the connecting channel. South approach walls upstream and downstream from the locks are approximately 2,200 feet long. Flare of the upstream wall is 1 foot in 10, while that of the downstream wall is 1 foot in 12.

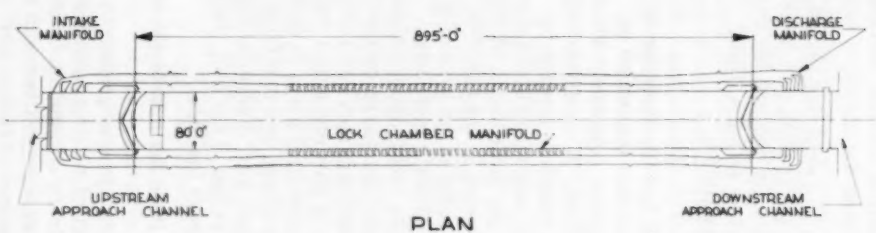
The two Beauharnois locks are very similar in design, the one appreciable difference being due to the provision of double miter gates at the lower lock while the upper lock has single gates. Extra gates at the lower lock are intended to be used as standby units and may be used as replacements for gates of the upper lock if necessary. Lock chambers are 80 feet wide by 895 feet long between miter gate pintle center lines, the latter distance being measured between inner miter gate pintles at the lower lock. Breast walls at the upper ends of the locks are 40 feet downstream from upper gate pintles to prevent ships with forward-sloping prows from damaging the gates. The lift at the site, averaging about 80 feet, is approximately evenly divided between locks, although the lift at either lock may vary by nearly 10 feet due to water level fluctuations in Lake St. Louis and the power canal.

The general arrangement of the filling and emptying system developed for these locks with the aid of model studies is indicated in Plate 4. The design is of the side wall type comprising longitudinal wall culverts with short laterals, or ports, opening into the lock chamber walls, and with intake and discharge ports located in the faces of lock approach walls. Wall culverts are 12 feet wide by 14 feet high at the filling and emptying valves expanding to a cross sectional area of 305.5 square feet between valves.

Since nearly all excavation for the locks had to be made in a very hard and abrasive sandstone, it was necessary, for the sake of economy, to keep lock walls as thin as possible. For this reason wall culverts in their enlarged section were made as high and narrow as it was considered practicable. Inner walls between the culverts and lock chamber had to be designed similar to a haunched beam section with a thickness of 10 feet at the lock floor and 14 feet at the tops of the culverts. Outer culvert walls are in the form of concrete slabs anchored to excavated rock walls. The enlarged culvert sections are 25 feet high with a top width of 9' - 6" and a width of 13' - 6" in their lower 9 feet of height. Lock chamber ports, being located at lock floor level, are 10 feet long.

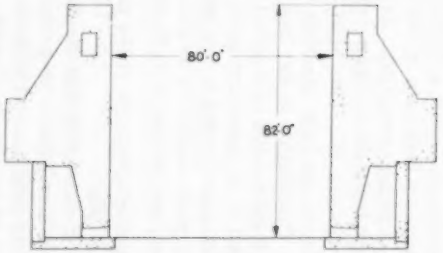
Because of their relatively short length, it was deemed advisable to design ports with a narrow throat section in order to control the direction of jets issuing into the lock chamber during a filling operation. For this reason, as well as to distribute port discharge along lock walls as evenly as possible, it was decided to use 40 small ports on each side of the chamber. Port throats are 3 feet, 6 inches high by 1 foot, 3 1/2 inches wide, giving a total

PLATE 4




PLAN

ELEVATION




PROTOTYPE CROSS-SECTION

THE ST LAWRENCE SEAWAY AUTHORITY

UPPER BEAUHARNOIS LOCK
HYDRAULIC SYSTEM

throat area of 180.8 square feet in each lock wall. A spacing of 10 feet 1 1/2 inches is used for ports in order to fit them into a lock wall with monolith joints 40 feet, 6 inches apart.

Information gained from previous models for Canadian Seaway locks was used extensively in making up a preliminary hydraulic design for the Beauharnois locks. The design modifications tested in previous models, and the improvements in lock performance gained thereby, are as follows:

1. Enlargement of the wall culverts between filling and emptying valves.
   It was found that enlarging the culverts, thus decreasing flow velocities past the ports, greatly improved flow distribution through the ports. This resulted in reduced surge in the lock chamber during a filling operation and decreased lock filling and emptying times.

2. Unsymmetrical venturi-shaped ports.
   Tests made with symmetrical ports indicated that the direction of jets from the ports during a lock filling operation was influenced by flow in the culverts. Jet direction had a downstream component which set up a downstream flow along the floor of the lock. A compensating upstream flow along the surface of water in the chamber exerted upstream force on a ship in the lock. In addition it was found that a surge was set up in the lock chamber due to a shift in the distribution of port discharge during a filling operation. Inertia of water in the culverts caused a high rate of discharge from upstream ports at the beginning of a filling operation. As flow velocities in the culverts increased, flow from downstream ports built up rapidly while that from upstream ports fell off. This shift in the distribution of port discharge caused a ship in the lock chamber to be forced first downstream then upstream.
   Circulation in the lock chamber and the shift in port discharge distribution were counteracted by the use of unsymmetrical ports to direct flow from upstream ports in the downstream direction, and that from remaining ports in the upstream direction. In this way, discharge from all ports tended to be concentrated in the mid-section of the lock chamber, and to flow in both direction toward the ends of the lock.

3. Center of gravity of the ports shifted upstream from the mid-length of the lock
   It was found that discharge from downstream ports was greater than that from upstream ports during the greater part of a filling operation. This unbalance of flow was reduced, but not entirely eliminated, by the use of enlarged culverts. By shifting the port arrangement some 20 to 30 feet upstream from a centered position in the lock chamber, port inflow was balanced about the lateral center line of the lock.

Unsymmetrical ports designed for Canadian Seaway locks have one side perpendicular to the lock center line while the other side is flared. In the Beauharnois lock model, best results were obtained with the twelve upstream ports in each chamber wall flared downstream, while all remaining ports are flared upstream. Shifting the port arrangement nearly 23 feet upstream from a centered position in the lock was found to give satisfactory results in the model.

In addition to the various port and culvert arrangements studied in the Beauharnois lock model, an investigation was made of the effect of ship winch operation on hawser forces developed during a lock filling operation.

This investigation indicated that if a ship can be held against the lock wall during a filling operation, ship motion is damped out by friction with the wall, and there is little danger of a build up in hawser tension. If not held against the wall a ship may be caused to oscillate longitudinally in the lock chamber due to improper winch operation, or to a wave disturbance in the lock. As in the case of ships moored in certain harbours which are subject to swell or wave action, any attempt to control ship motion by increasing tension in all hawsers simultaneously may lead to severe overstress in cables and winches unless automatic winches are used.

From model test results it is estimated that the locks, under average lift conditions, can be filled in 6 minutes and emptied in 7.3 minutes. While the model indicated that a large lake freighter type of vessel can be handled with ease during a 6 minute filling operation, local turbulence in the lock chamber may make it difficult to control a smaller vessel at maximum filling rates. A filling time of about 7.5 minutes should be suitable for vessels of all types.

## Regulating Works for the Channel Between Locks

Some interesting hydraulic problems were encountered in the design of regulating works to control water level in the short channel between locks at Beauharnois. As previously mentioned, this channel has a surface area of little more than 26 acres so that a single lockage operation could change the water level by more than 2.5 feet.

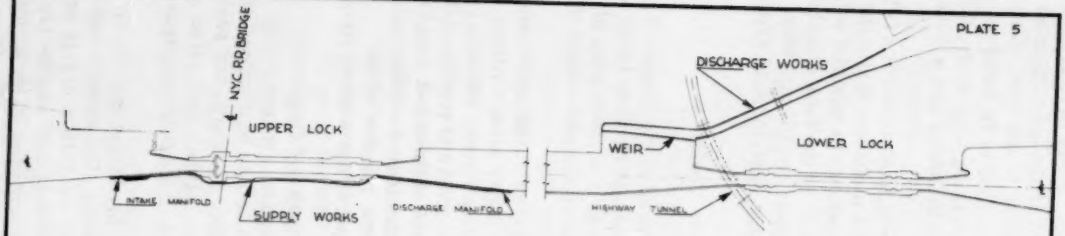
Calculations were first made to determine the height of surge which might be caused in the channel by an empty operation at the upper lock, a filling operation at the lower lock, or by both of these operations made simultaneously. These calculations indicated that, due to the short length of the channel, there was little danger that a surge height of more than about 2 feet could be built up by lockage operations. This height was not considered objectionable since surge heights of more than 2.5 feet occur in some reaches of the Welland canal during the course of normal lockage operations with no apparent detrimental effect on shipping.

Theoritical studies of possible lockage sequence, both for single and double locks, and their effect on water level in the channel between locks showed that a maximum regulating inflow of 850 c.f.s. would be required to maintain a safe depth for navigation. Partly as a safety measure, and partly to compensate for losses such as seepage and leakage through lock gates and valves, it was decided that the supply works should have a minimum capacity of approximately 1000 c.f.s.

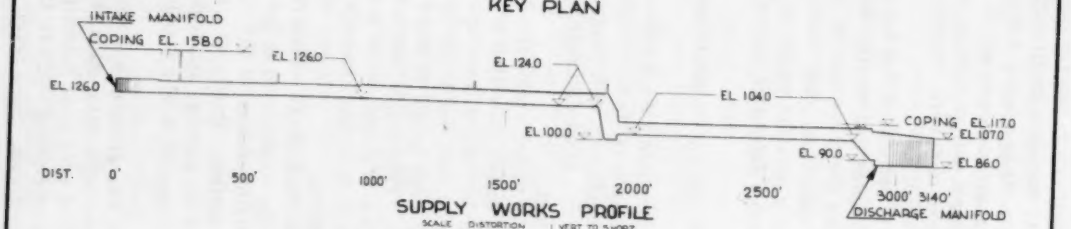
Preliminary consideration was given to a scheme for taking this water from the power canal adjacent to the navigation channel. The scheme was abandoned, however, because of very poor subsurface conditions in this area. The power dyke was found to be resting on 12 to 15 feet of soft marine clay requiring very flat excavated slopes. Bore holes encountered water under considerable pressure in the weathered bedrock surface layer. Due to these conditions there was considerable risk in breaching the power dyke and, although a much greater length of conduit was required, it was considered preferable to obtain water from the upstream lock approach channel.

The supply system, as shown in Plate 5, consists of an intake manifold with 10 ports in the upstream south approach wall of the upper lock, a conduit located along the south side of the lock, and a discharge manifold with

PLATE 5




KEY PLAN




SUPPLY WORKS PROFILE

SCALE DISTORTION 1 VERT TO 5 HORZ

DISCHARGE WORKS PROFILE

SCALE DISTORTION 1 VERT TO 5 HORZ

THE ST LAWRENCE SEAWAY AUTHORITY

DETAILS OF REGULATING WORKS FOR THE CHANNEL BETWEEN LOCKS AT BEAUHARNOIS

15 ports in the channel wall below the lock. The conduit is built into lock approach walls, but for structural reasons, could not be fitted into walls of the lock itself. Flow is regulated by sector type valves located near the upstream end of the conduit. Intake and discharge manifolds are located well away from the ends of the lock to prevent cross current interference with ships as they enter and leave the lock chamber. To obtain reasonably uniform flow through intake ports, throat areas of these ports are increased toward the upstream end of the manifold. This same result is obtained at discharge ports by increasing culvert area in the upstream direction.

Discharge manifold design was made with particular care to prevent the setting up of cross currents in the navigation channel. Ports are 20 feet high to distribute flow over a large proportion of channel depth, and have a throat width of only 8.4 inches so that jets issuing into the channel will be diffused within a few feet of the channel wall. Design calculations indicate a maximum port exit velocity of 4.2 feet per second when total regulating flow is 1000 c.f.s.

To reduce excavation costs, the conduit between the intake manifold and the lower end of the lock is placed at rock surface level. This upper section of conduit discharges into a drop structure from which flow enters a second section of conduit located below lower coping level. Design of the upper conduit section was complicated by the fact that water level in the power canal fluctuates over a range of nearly 10 feet, and maximum regulating flow is required when water level, and therefore, discharge head, is at a minimum. In order to simplify the design of the drop structure and lower section of culvert as well as the discharge manifold, it was necessary to make the conduit head-discharge curve as flat as possible. Pressures in the upper conduit had to be kept as low as possible to minimize leakage into the fill behind lock wall monoliths.

In the final design an attempt was made to satisfy all of the above mentioned requirements as nearly as possible. The upstream 756 feet of conduit is 7 feet high by 9 feet wide, with zero slope. At the downstream end of this section there is a sudden expansion to a section 8.5 feet by 10 feet in area. This larger conduit extends downstream to the drop structure, a distance of 913 feet, has a slope of 0.262 per cent, and is designed to carry 900 c.f.s. when flowing to a depth of 9 feet. V - shaped baffles, extending one foot downward from the roof, are designed to retard flow when discharge exceeds 900 c.f.s.

The drop structure has a length of 70 feet, its floor being 24 feet below the invert of the upstream conduit, and 4 feet below the floor of the exit conduit. The depth below floor level of the exit conduit is to be drained, when necessary, through a 6-inch pipe leading to the face of the lock approach channel wall. Dimensions of the drop structure are such that it provides a free water surface at all times for the release of entrained air from water before it enters the lower conduit.

The exit conduit, 906 feet in length, is 8 feet wide by 9 feet high with zero slope, and curves into the approach wall just below the lock. Flow from this conduit passes through a transition section 82 feet long into the discharge manifold which has a total length of 240 feet. Energy in high velocity flow from the conduit is dissipated in a hydraulic jump in the transition which ends 51 feet upstream from the first discharge port to prevent turbulence due to the jump from interfering with port discharge.

As previously mentioned, the discharge manifold has 15 ports which are

spaced 12 feet apart. The upstream 71 feet of conduit in the manifold, beginning 51 feet upstream from the first port and ending opposite the fifth port, has a uniform width of 10 feet. Transitions located between the fifth and sixth ports and between the tenth and eleventh ports, reduce conduit width to 8 feet and 6 feet respectively. Height of the conduit is 25 feet at its upstream end and 21 feet at its downstream end, the roof being sloped longitudinally to prevent entrapment of air as water level in the navigation channel rises.

With regulating valves located near the intake ports, water level in the entire conduit system will rise and fall as the valves are opened or closed. In addition, water level in the discharge manifold and lower conduit section will be affected by water level fluctuations in the navigation channel. For this reason the design provides for an ample number of air vents.

Design calculations for the supply system indicate a minimum discharge capacity of 950 c.f.s. with a head of 14.4 feet on the intake ports, and a maximum capacity of 1250 c.f.s. corresponding to a head of 23.9 feet. With a discharge of 1250 c.f.s. maximum hydraulic grade line elevation for the section of conduit located behind locks wall monoliths will be 5 feet above the roof of the conduit.

As previously mentioned, the discharge works for regulation of water levels in the channel between locks consists of a side-channel spillway with chute located north of the lower lock. For the control section a long unregulated fixed crest weir was chosen because of its simplicity and reliability. Designed as an ogee section for a discharge head of 2.5 feet, 700 feet of crest is set at an elevation 4 feet below the tops of upstream miter gates at the lower lock. The upstream 12 feet of crest is designed as a circular arc with a radius of 3.5 feet, and is 1.7 feet lower than the ogee section.

The discharge channel has a width of 42 feet at its upstream end, increasing to 54 feet at the downstream end of the weir, with corresponding depths of 7 feet and 17 feet below weir crest level. This channel is in the form of an unlined rock cut.

The concrete-lined chute has a slope of 2 per cent, a width of 50 feet, and a length of roughly 1,500 feet. Its downstream 375 feet of length is flared to accommodate a flip bucket 100 feet long located just offshore in Lake St. Louis. In order to prevent erosion of fill in the downstream lock approach by chute discharge, the chute is curved away from the lock through an angle of 28 degrees at its upstream end, the center line radius of curvature being approximately 370 feet. The curved section, with a total length of 190 feet, has been given zero slope to obtain sub-critical flow through its entire length. Also, in its curved section, the chute passes over the highway tunnel on cellular concrete cribs.

The reason for setting a 12-ft. section of the weir at a lower elevation than the remaining weir length is chiefly to prevent accumulations of ice in the discharge system during cold weather, particularly during slack periods in shipping. If water in the channel were allowed to remain at weir crest level for extended periods, it would spill over periodically, due to wind or surge action, which would result in a build-up of ice on the weir as well as in the side channel and chute. The short depressed section of weir is intended to draw water level down below the main regulating weir crest. It is expected to supply sufficient flow to prevent ice accumulation in the discharge channel and chute.

In addition to its purpose of channel water level regulation, the discharge works will have a damping effect on surges caused by emptying operations at

the upper lock. Its maximum capacity of nearly 15,000 c.f.s. provides a safety factor against the overtopping of gates at the lower lock.

Regulating works for the channel between locks at Beauharnois are designed chiefly to limit water level fluctuations in the channel caused by changes in water levels in Lake St. Louis and the power canal, rather than to control fluctuations due to individual lockage operations. Channel depth below crest level of the main discharge control weir is 35.2 feet, or 6.2 feet deeper than that called for in the adopted Seaway standards for channels with rock floors. In calculating the capacity of supply works required for channel regulation it was assumed that water level would be allowed to drop 4.2 feet below weir crest level, which is a much greater drawdown than would result from a filling operation at the lower lock. Due to the short distance between locks, emptying operations at the upper lock alternated with filling operations at the lower lock may be depended upon to partially balance water supply and demand. When lift at the upper lock is greater than that at the lower lock, which will occur approximately 50 per cent of the time, this condition will be compensated by high water levels in the channel between locks together with discharge over the regulating weir. When lift at the lower lock is higher than that at the upper lock, water level in the connecting channel will be regulated below weir crest level, and flow from the regulating supply works will be depended upon to balance lockage demand.

The general arrangement of Seaway locks and channels now under construction at the Beauharnois site was selected as that best adapted to present and future Seaway needs. It was recognized from the beginning that this arrangement would require careful treatment with regard to hydraulic design in order to provide lockage facilities with operating performance equal to that at other Seaway lock installations. Every effort has been made to insure that this result will be obtained.

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by Robert E. Hickson . . . . . 1785-11

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# CELLULAR COFFERDAMS AND DOCKS[a]

## Closure by E. M. Cummings

E. M. CUMMINGS,[1] M. ASCE.—The discussion of this paper by Mr. Heyman is appreciated since it suggests points in the original paper which can be clarified. Heyman suggests additional tests on cellular cofferdams to determine where in the fill the actual plane of failure occurs. In connection with the model studies upon which the paper was based, a preliminary model was constructed to determine whether there is any definite plane of failure. This model was 12" x 12", 1'-0" high. It was filled with white sand, except that horizontal thin layers of black sand were placed at about 1-1/2 inch intervals. Thin transparent plastic strips 3/4" wide were used to simulate sheet piles, and were retained by two light exterior wood frames. When this model was tilted by a lateral force, the black streaks of sand showed clearly there was no definite plane of rupture. The flow of the material was similar to that of a plastic mass, and the black sand layers remained approximately horizontal, although there was a tendency of the sand to "pile up" on the side on which the force was applied, and to subside on the opposite side of the cell. This was also observed in the larger models described in the original paper.

Heyman notes that the angle of internal friction of the fill material was determined by making the cell slide on a base comprised of the same material, and states that this procedure was unjustified, since the angle of internal friction was determined from the test results themselves, and not by an independent method. It is pointed out, however, that the angle of internal friction was checked by measuring the natural slope of repose, the latter being only about 2 degrees less than the angle determined by making the cell slide. It is generally accepted that for non-cohesive soil, the natural slope of repose defines the lower boundary of the angle of internal friction.

It is considered significant that the force which caused the model cell to slide, also caused it to tilt, when the force was applied at the critical height $\bar{y}$ above the base. This fact demonstrated that the tilting of a cell is identified with horizontal sliding of the fill in the cell. This is the basis of the theory presented.

Heyman points out that the writer discussed the Terzaghi(1) theory of failure on a vertical neutral plane, on the basis of the Rankine coefficient of pressure, although Terzaghi later accepted Krynine's coefficient. The reason that the Rankine coefficient was discussed is that there are at least three texts (Ref. 2, 3, and 4) and probably more, which use only the Rankine coefficient in their presentation of the Terzaghi theory. This, combined with the

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a. Proc. Paper 1366, September, 1957, by E. M. Cummings.

1. Mgr. of Sales, Bethlehem Steel Co., Bethlehem, Pa.

fact that engineers are using the Rankine coefficient in designing with the Terzaghi theory, indicated that a discussion on this basis was pertinent.

Assuming a proportion of b = .85 h, Heyman puts Eq. (6) for the moment of resistance of the sand fill in the same form as Terzaghi's equation, and thus derives a coefficient of pressure M. He then explores this coefficient for values of $\phi$ from 0 to 90 degrees, thereby subjecting it to the same rigorous treatment that the writer used in investigating the Terzaghi coefficient. In principle there can be no objection to this. But in doing so Heyman assumed conditions beyond those for which Eq. (6) was derived. The equation was derived from the condition shown in Fig. 8(c). Fig. 1 herewith shows the condition for $\phi = 60^0$, and it is obvious that Eq. (6) could not be derived from it.

Heyman's graph plotting $\phi$ against C tan $\phi$ is not correct for the coefficient M for values of $\phi$ greater than $45^0$. The theory presented in the original paper is based upon resistance to horizontal shear which is proportional to tan $\phi$, which in turn increases as $\phi$ increases. Thus there is no reversal in the theoretical resistance for higher values of $\phi$ as indicated in Heyman's graph for coefficient M. From a practical standpoint, of course, there can hardly be any fill material with $\phi$ greater than $45^0$.

Mr. Erzen discusses the use of 3/4 inch thick vertical planks used to simulate sheet piles in the model cells. The model cells were made with wood staves 5/16" x 1-1/2" x 2'-0" as mentioned on Page 5 of the paper, and not with 3/4" planks. He introduces consideration of the bending strength of the model sheet piles in his analysis. If bending had been a factor, it would have resulted in higher total resistance than the horizontal shear value of the fill, which was the greatest resistance developed by the model cells. He is probably influenced by the photograph shown in Fig. 3(b). However, this photograph was taken after extreme tilting and distortion had taken place, far beyond the point at which the cell developed its maximum resistance, which occurred after about 1-1/2" of movement of the top. Fig. 3(b) shows about 6-1/2" of movement.

Erzen presents a new theory for lateral resistance to tilting which is illustrated in his Fig. 2. He assumes that sliding will occur on a plane which makes an angle of $45^0 + \phi/2$ with the vertical. Fig. 2 herewith reproduces a portion of Erzen's Fig. 2 with modifications for the purpose of discussion. He assumes that the lateral resistance against sliding on plane LB is equal to the Rankine passive pressure of wall JL less the pressure of wall EB. However, the Rankine theory proves that the plane of least resistance is $45^0 + \phi/2$ only when the surface of the ground is horizontal for an indefinite distance, in which case the plane LBD of Fig. 2 is the plane of least resistance. For any other condition the plane of rupture is not inclined at $45^0 + \phi/2$ to the vertical.

Fig. 3 herewith determines the plane of least resistance by the Culmann method. LB is plotted at the angle below the horizontal, and planes L1 to L5 are trial planes of rupture. Line AB is the smallest intercept between LB and the Culmann curve, and its length gives the resistance to sliding on the horizontal plane L5. The least resistance to sliding is therefore on a horizontal plane, and not the plane at $45^0 + \phi/2$ as Erzen assumes.

## REFERENCES

1. Terzaghi, Karl, "Stability and Stiffness of Cellular Cofferdams", Transactions ASCE Vol. 110, 1945, p. 1083.

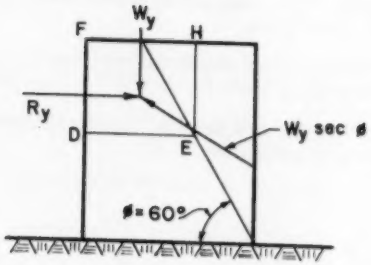


Fig. 1

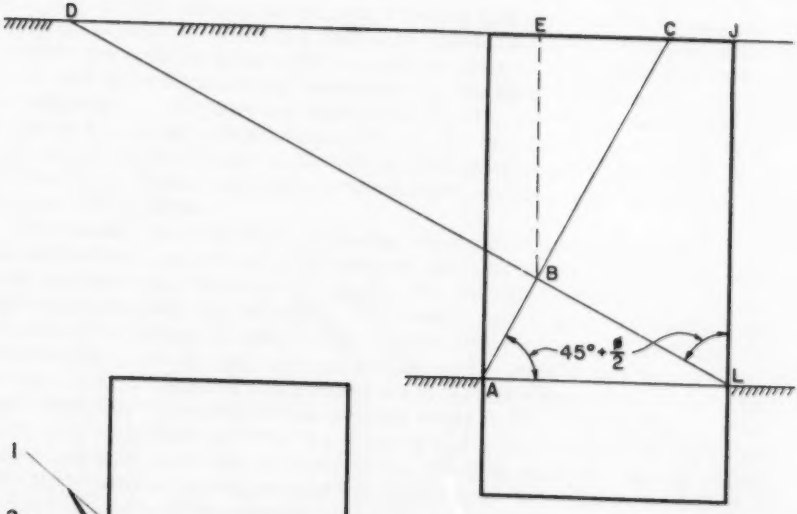


Fig. 2

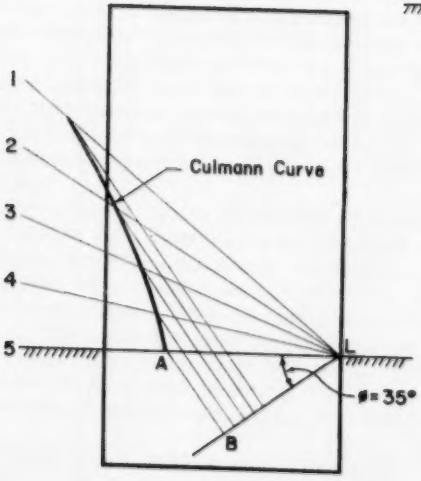


Fig. 3

2. Dunham, Clarence W., "Foundations of Structures", p. 450, McGraw-Hill Book Company, Mc. New York.

3. Anderson, Paul, "Substructure Analysis and Design", p. 177, The Ronald Press Company, New York.

4. "Shore Protection Planning and Design". Beach Erosion Board. U. S. Govt. Printing Office, Washington, D. C.

# SHIPBOARD HYDRAULIC BREAKWATER[a]

Discussion by John B. Herbich

JOHN B. HERBICH,[1] A. M. ASCE.—Some work had been done on three-dimensional models at the University of Minnesota(1) and California.(4) These limited tests seem to indicate that waves tend to refract and augment. This is true in case of a hydraulic breakwater, when jets are directed against the oncoming waves. One of the ways of avoiding this would be to employ a pneumatic breakwater in which the "surface" currents are generated by air bubbles rising to the surface against the current, as well as in the opposite direction. Additional tests of pneumatic breakwater on three-dimensional models would be required to verify this and determine the extent of the protected area produced by a pneumatic breakwater. Pneumatic breakwater tests have been carried out by the Kyushu University in prototype installations. It would be interesting to learn the results of these tests, which are on the largest scale carried out anywhere.

The method used by Dilley in finding transmitted wave height should be commented on. According to the author, only five of the lowest wave heights were averaged to determine the height of transmitted waves. It is felt that this was inadequate and is reflected at least to some extent in considerable scatter of the data presented in Fig. 6. The determination of the transmitted wave height is not an easy matter, and for an accurate evaluation of the transmitted wave, the heights of all waves for a certain representative length of time should be measured and an average value taken. This average value gives only an indication of the wave height and a complete analysis of wave height might be desirable in some cases. Such an analysis may be lengthy and cumbersome. A simpler method was devised for tests described in (1) in which lines were drawn through 16 to 20 representative groups of waves, and these values measured and averaged instead of averaging some 400 wave heights.

In his paper, Dilley states that the graphs of wave attenuation as a function of jet-flow rate show that in the range of wave heights tested and within the range of experimental error, the height of the waves at a given ratio of wavelength to water depth does not affect the flow rate required to cause a given wave attenuation. This statement seems unjustified and not supported by the data. The tests described in (1) indicated that wave height has appreciable effect on discharge requirements and considerable effect on power requirements. Typical data on power requirements are shown in Fig. 1 for three wave length-water depth ratios. Considering the curve for length depth ratio

a. Proc. Paper 1569, March, 1958, by R. A. Dilley.

1. Asst. Prof., Dept. of Civ. Eng., Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, Pa.

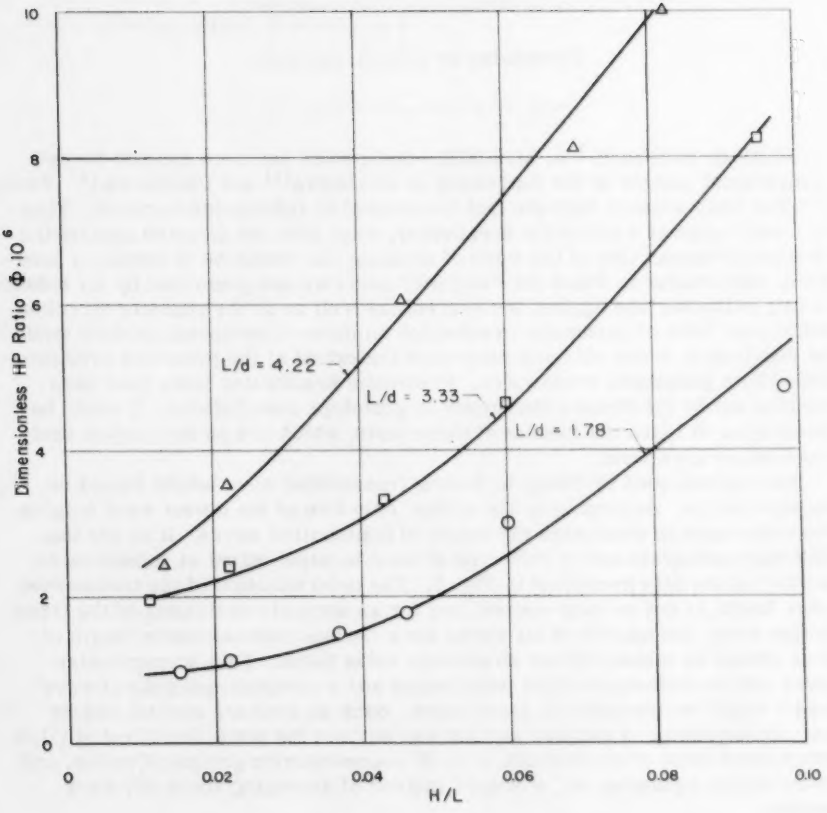


Fig. 1—Horsepower Ratio as a Function of Wave Steepness

of 3.33 it may be noted that for an increase in wave steepness from 0.02 to 0.08, a factor of four, the required horsepower ratio is measured by a factor of about three. The dimensionless horsepower ratio is described as

$$\phi = \frac{\text{Horsepower per lineal foot of breakwater}}{\rho\, g^{3/2}\, L^{5/2}}$$

The author is to be complimented for performing difficult experiments and for publishing results of his tests.

## REFERENCES

1. Herbich, J. B., Ziegler, J., and Bowers, C. E., Experimental Studies of Hydraulic Breakwaters, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Project Report No. 51 June 1951.
2. Straub, L. G., Herbich, J. B., Bowers, C. E., An Experimental Study of Hydraulic Breakwaters. Paper presented at the 6th International Conference on Coastal Engineering, University of Florida, December, 1957.
3. Anon. On the Study of a Pneumatic Breakwater. Reports of Research Institute for Applied Mechanics, Kyushu University, 1956.
4. Snyder, C. M., Model Hydraulic Breakwater Studies; Wave Research Laboratory, University of California, 1957.

# STABILITY OF COASTAL INLETS[a]

Discussion by Robert E. Hickson

ROBERT E. HICKSON,[1] M. ASCE.—The subject the authors have discussed in their paper is a most interesting one and in many cases is not susceptible of solution with the certainty and accuracy that other hydraulic problems may be solved. As stated, there are involved "ten to twenty or more variables" and many combinations of these.

The authors discuss inlets which are at the mouths of rivers and others which are openings through barrier bars or spits built generally by littoral currents or wave action or combinations of these. The controlling influences in the two general cases are very different, especially where the river flow is relatively large.

This discussion and data submitted will be related principally to the cases affected by river flow, with which the writer has had more experience and has made tidal and current surveys to secure basic data.

In the case of upland freshwater flow and a substantial tidal range, the discharge of the river flow to sea is entirely stopped during a major part of the flood tide phase. Then during the succeeding ebb the outflow of fresh water has to be nearly double the headwater flow of the river to make up for the time lost during the flood tide phase. Where the river flow constitutes a considerable part of the prism to be discharged, the ebb flow and velocities accordingly exceed those of the flood by substantial amounts. Another consideration is that in cases of the mixed type of tide with large diurnal inequality the maximum outflow and tidal prism discharged is on the great ebb running down to the lower low, following the higher high tide. This is characteristic of the North Pacific Coast tides, and others.

The density current prevailing in the inlet for an hour or two at low water turn of the tide also introduces a complication making mathematical solution more difficult.

Observations were made during a comprehensive current survey at mouth of the Columbia near the "gorge" section in 1932, at three different stages of the river (low, intermediate and high). Velocity measurements were made at five points in the verticles, at five boat stations in the section, every 20 minutes continuously day and night for periods of several days for each river stage. (Report was made to Chief of Engineers U. S. A. under the title Columbia River Current Survey 1932-33).

The following tables are taken from the above survey report. Table 1 covers maximum, minimum and mean velocities at the three river stages on

a. Proc. Paper 1644, May, 1958, by Bruun & Gerritsen.

1. U. S. Army Engr. Retired, Portland, Ore., formerly member of Committee on Tidal Hydrs.

Table 1

COLUMBIA RIVER

Mean Velocities at Strength of Tides
For Section at Clatsop Spit

| River Stage | | Velocities ft. per sec. Range | Average |
|---|---|---|---|
| Low; September: | | | |
| Ebbs | (7 tides) | 4.35 to 5.39 | 4.85 |
| Floods | (7 tides) | 4.09 to 4.96 | 4.51 |
| Intermediate; April: | | | |
| Ebbs | (3 tides) | 4.48 to 5.65 | 5.06 |
| Floods | (8 tides) | 3.38 to 4.12 | 3.85 |
| High; May - June: | | | |
| Ebbs | (17 tides) | 4.57 to 7.20 | 5.58 |
| Floods | (16 tides) | 1.95 to 4.32 | 3.36 |

ebb and flow tides, and Table 2 shows percentages of time various bottom velocities prevailed during the three river stages.

The chart, Fig. 1, shows graphically how the bottom velocities flood and ebb were affected by the variations in headwater flow for 5%, 10% and 40% of the total time of observations.

Table 2

COLUMBIA RIVER

Percentages of Time Bottom *Velocities
Exceed 1, 2, 3, & 4 ft. per sec.

| Stage | Velocity 1' | Velocity 2' | Velocity 3' | Velocity 4' |
|---|---|---|---|---|
| | Percent of Time | | | |
| Low: | | | | |
| Ebbs | 37.6 | 19.8 | 3.3 | 0.0 |
| Floods | 40.2 | 22.0 | 4.5 | 0.4 |
| Intermediate: | | | | |
| Ebbs | 36.8 | 18.2 | 2.3 | 0.2 |
| Floods | 34.0 | 16.3 | 2.3 | 1.0 |
| High: | | | | |
| Ebbs | 44.6 | 25.5 | 9.0 | 3.5 |
| Floods | 31.2 | 14.5 | 2.5 | 0.0 |

*Velocities measured at 9/10 depth

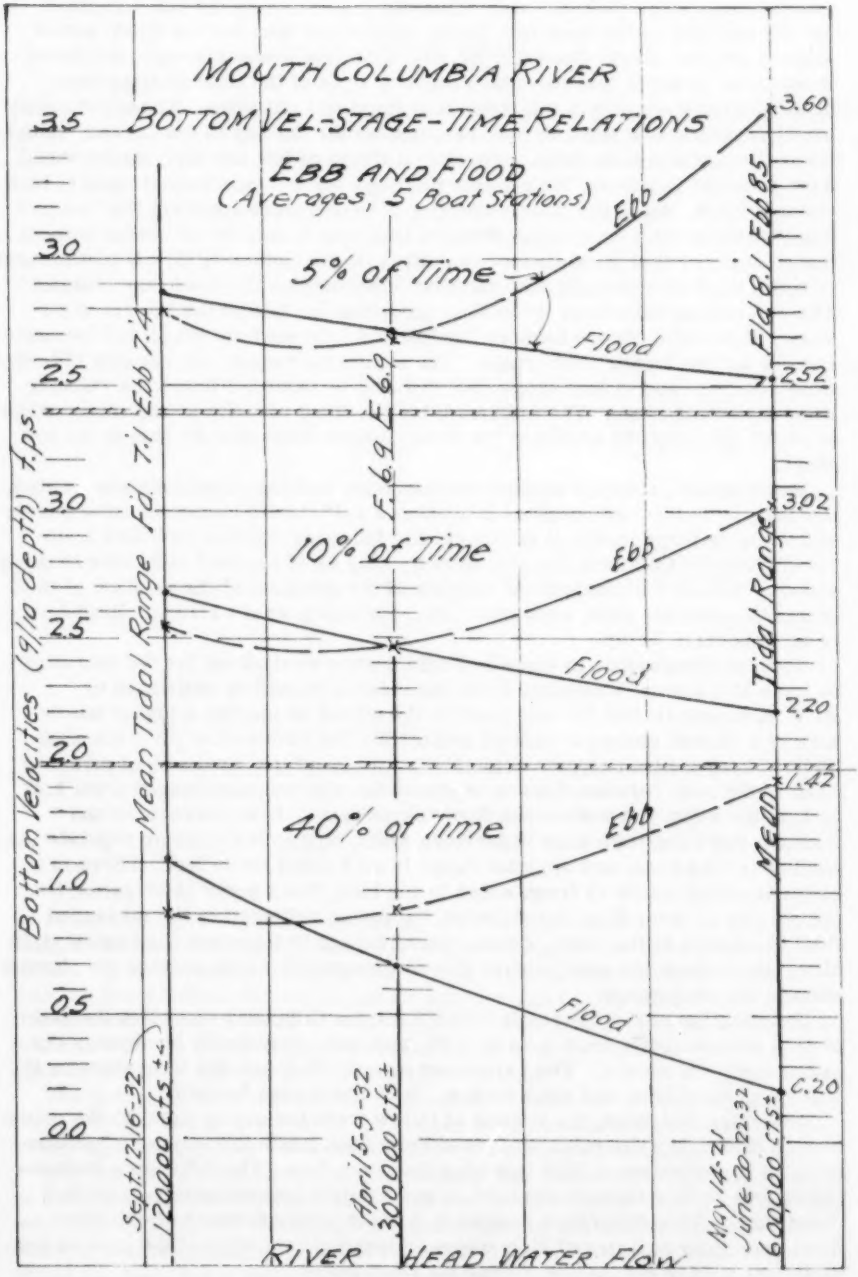
MOUTH COLUMBIA RIVER
BOTTOM VEL.-STAGE-TIME RELATIONS
EBB AND FLOOD
(Averages, 5 Boat Stations)
5% of Time
10% of Time
40% of Time
Ebb
Flood
Bottom Velocities (9/10 depth) f.p.s.
Mean Tidal Range Fd. 7.1 Ebb 7.4
Fd. 6.9 Ebb 6.9
Fld 8.1 Ebb 8.5
Tidal Range
Mean
3.60
2.52
3.02
2.20
1.42
0.20
3.5
3.0
2.5
2.0
1.0
0.5
0.0
Sept. 12-16-32
120000 cfs ±
April 5-9-32
300000 cfs ±
May 14-21
June 20-22-32
600000 cfs ±
RIVER HEAD WATER FLOW

FIG. 1.

It will be noted that the velocity curves on Fig. 1 show a slight dip at the intermediate stage (300,000 cfs). This dip is greater on the ebb velocities for 5% and 10% of the time (the higher velocities) than for the flood, and is slightly greater on the flood for the 40% time (the lower average velocities). It will also be noted that the tidal ranges at time of the intermediate stage observations were only 6.9 feet for both flood and ebb tides. These relatively small ranges are at least in part responsible for the sag in the curves. Ranges more comparable with those occurring at times of low and high stages would have resulted in raising the velocity plottings for the intermediate stage, both ebb and flood. As to the flood velocities it is therefore apparent that the plottings would be on a practically straight line, and it may be concluded without material error that flood bottom velocities on the Columbia for all percentages of time, vary on a straight line relation, inversely as the head flow changes. The ebb bottom velocities, even after adjusting for tide at the intermediate stage, show little change between the low and intermediate stage, but increase rapidly for the higher river stage. The maximum bottom ebb velocity (5% of time), in this case at high stage (600,000 cfs) is about 1.3 times the velocity at intermediate stage. The maximum bottom flood velocity at high river stage is shown as about .92 of that at the intermediate stage and .87 that at the low stage.

Since actual prototype measurements under various conditions are limited, due largely to the cost involved in making a sufficiently comprehensive survey and study on large inlets, it is thought the following figures, in Table 3, derived from the Columbia Current Survey, may be of interest and value to those working toward a mathematical solution of the problem of the relation of tidal prism to sectional area, velocities, etc., for cases where river or head flow is an important factor.

Figures comparable to those in Table 3 were worked out for the entrance to Lake Maracaibo, Venezuela from the volume of outflow estimated by E. F. Robinson in 1938-39 and used by the writer in making a layout and design of a 36 foot entrance channel project for the Venezuelan government in 1947. From the limited data available it was found that the ratio of prism to area in the inlet between Zapara Island at San Carlos point ranged from 1.23 to 1.86 for a dry and wet season flow respectively. Lake Maracaibo and Tablazo Bay constitute huge reservoirs which tend to stabilize or regulate the outflow in this case, and the tidal range is only about three feet. There is a large seasonal inflow of fresh water to the lake, from some 15 streams, but the volume of river flow is not known, except as reflected in the measured tidal discharge at the inlet. Jetties were laid out to intercept the littoral drift along shore from the east (a wave driven movement) which shoaled the channel outside the inlet gorge.

Examination of the figures in Table 3 for the Columbia show that the ratio of P/A varies widely from 1.24 to 3.03, with relatively small changes in the cross sectional area A. The variations in ratio P/A are due to changes in the volume of head flow, and tidal ranges. Both are potent factors.

As before indicated, the volume of inflow from the sea is equal to the prism at high tide minus the volume of river head flow, while the volume of outflow is equal to the prism at high tide plus the river flow. The difference between inflow and outflow through the inlet is accordingly approximately equal to 2 (head flow). This difference results in a great preponderance of ebb over flood velocities at times of high stages in large rivers where tidal ranges are sufficient to stop the outflow during the flood phase.

Table 3

COLUMBIA RIVER*

| Date 1932 | Head Flow $10^3$ cfs. | Ebb Range Ft. | Metered Tdl. Prism $10^3$ Ac.Ft. P | Area (1) M.S.L. $10^3$ Sq. Ft. A | Ratio P/A | Vel. in Sec. Mean Fps. | Vel. in Sec. Maximum Ave. in Verticle |
|---|---|---|---|---|---|---|---|
| 4/6 | 316.0 | 5.8 | 709.8 | 430.5 | 1.65 | 3.14 | 4.92 |
| 4/8 | 320.0 | 8.7 | 879.1 | 430.5 | 2.04 | 3.27 | 5.65 |
| 4/8 | 320.0 | 4.4 | 535.2 | 430.5 | 1.24 | 2.57 | 4.12 |
| 5/14 | 512.0 | 5.0 | 776.3 | 428.2 | 1.83 | 2.89 | 4.62 |
| 5/17 | 579.0 | 10.0 | 1152.0 | 428.2 | 2.70 | 3.49 | 6.12 |
| 5/21 | 554.0 | 12.1 | 1295.8 | 428.2 | 3.03 | 4.20 | 6.52 |
| 6/22 | 564.0 | 9.2 | 1095.9 | 428.2 | 2.55 | 3.75 | 6.27 |
| 9/12 | 131.0 | 5.3 | 579.7 | 450.0 | 1.29 | 2.80 | 4.35 |
| 9/13 | 127.0 | 8.8 | 789.0 | 450.0 | 1.75 | 3.41 | 4.99 |
| 9/15 | 121.0 | 7.9 | 704.5 | 450.0 | 1.56 | 3.11 | 4.74 |

(1) Area at mean sea level from 3 surveys

* The figures here included, are taken from only a few out of the observations made in April, May, June and September in the current survey of 1932. In total observations were made on 32 ebb and 31 flood tides involving over 5600 individual current meter recordings.

Flood bottom velocities from density currents, at the last of the ebb cycle, complicate the problem, but since for medium to high river flows the ebb bottom velocities predominate and at higher river stages are much greater than the flood bottom velocities for all percentages of time, the flood velocities are not shown in Table 3. Reference to Table 2 will show the general relation of ebb and flood bottom velocities as affected by river stages. These figures are from the complete set of observations.

The foregoing will indicate the difficulties involved in attempting to set up a formula for use in arriving at the proper (self maintaining) area of an inlet where appreciable river flow and density currents are involved.

It is thought we can agree, as Eaton says, that large inlets will have a larger ratio of prism to sectional area, than the smaller ones. Rocky shores may not be the full answer however if the major part of the bottom is sandy material. The Columbia, in granular material (relatively coarse sand in the gorge section), has a ratio under normal conditions considerably larger than the average.

The maintenance of depths in an inlet is dependent upon the critical velocity on the bottom necessary to move the material existent in the inlet, particularly the bottom material. Affecting this critical velocity is to be considered the boundary of the effect of wave action. In moderate depth channels the effect of average waves may reach the bottom to stir the material, so that lesser bottom velocities will result in movement. The end effect of this is to produce proportionately greater depths in the small inlets for the same bottom velocity, and a smaller value of the ratio P/A. Conversely in the very deep inlets normal wave action may not reach the bottom to stir the materials, thus requiring a higher bottom velocity and a smaller section for stability. This agrees with the author's thoughts on page 37. If in large deep channels, the ratio of bottom velocity to the mean Vb/Vm is less than for smaller shallower channels, a condition of equilibrium requires that P/A be greater for the larger deeper inlets, or that the sectional area be relatively smaller.

Reference to Table 3 shows the effects of high head flow in the Columbia as well as large tidal ranges on the ratio of P/A. It also shows the effect of these on mean velocities in the gorge section and the maximum average in any verticle (6.52 fps). The maximum (at peak of ebb) measured discharge on other tides (not shown in the table) was 3,065,000 c.f.s. with a maximum average in a verticle of 7.2 fps. Peak measured velocity near the surface was 12.25 fps.

Using the O'Brien formula with prism measured on 9/13/1932 gives a cross section in fair agreement with the prototype. The ratio P/A under these conditions was 1.75. Under other conditions of river flow and tidal ranges the ratio varied widely as before stated and shown.

The curves on authors' Fig. 9 show graphically that the fundamental equation Q = AV holds for inlet channels as well as rivers. Contrary to this, it is known that many engineers in earlier days laid down the controlling jetties with a convergence toward the sea end. Such a layout often leads to trouble if the convergence is great, as the inner channel is then largely uncontrolled. It meanders out of the desired alignment, and shoals as a result of excessive width, or develops deep water at a location not satisfactory to navigation, with a shoal where the channel should be.

All in all, this subject of inlet size and maintenance is a most engaging one and much has been gained from studies of the somewhat limited prototype data presently available. It is agreed with the authors that we should not "try to tell nature how it is expected to behave . . ." Existing channels developed by natural flow over a long period of years in which time the effects of all the contending forces have been integrated by nature, as shown by surveys extending over many years, furnish compelling evidence as to what may be expected under the circumstances and conditions in any case. This evidence should be fully recognized and evaluated.

Every waterway is a problem in itself, differing from others in some respects, and while these differences are not always apparent, some of them may be vital to the problem at hand. It accordingly behooves the engineer to be very circumspect, and heed well the indicated answers nature may have provided.

The authors are to be commended for the comprehensive analysis and presentation made of this complex subject, and it is hoped that the foregoing discussion and data submitted may add something toward the solutions desired.

Paper 1789 September, 1958 WW 4

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# NAVIGATION ON THE COLUMBIA RIVER

Ray E. Holmes,[1] M. ASCE
(Proc. Paper 1789)

## SYNOPSIS

The object of this paper is to present the historical development of navigation on the Columbia River, including a description of navigational structures, maintenance procedures, past and present, and a discussion of possible trends. The report is confined to the Columbia River between its mouth and the Canadian border plus the lower portion of the Willamette River to Portland, Oregon, and the Snake River from its confluence with the Columbia to Lime Point, both of which are integral parts of the Columbia River system.

## A. Description

1. General. - The Columbia River Basin extends from the Continental Divide to the Pacific Ocean. The river rises in Columbia Lake in Canada, is 1,210 miles in length, and drains an area of 259,000 square miles. From its source in British Columbia, 75 miles north of the International Boundary, the Columbia River flows northwesterly for 200 miles, turns and flows almost due south for 260 miles, entering the United States near the northeastern corner of Washington state. From the International Boundary the river flows in a southerly direction 441 miles through the eastern part of Washington, then flows westward 309 miles between Oregon and Washington to the ocean. The Canadian portion of the basin, which lies in the southeasterly part of British Columbia, totals 39,500 square miles. The 219,500 square miles in the United States includes most of Idaho, most of Oregon and Washington, all of Montana west of the Continental Divide, and small areas in northern Nevada, northern Utah, and western Wyoming.

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1789 is part of the copyrighted Journal of the Waterways and Harbor Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 4, September, 1958.

1. Chf., Rivers & Harbors Section, Portland District, Corps of Engrs., U. S. Dept. of the Army, Portland, Ore.

2. Tributaries. - Snake River, the largest tributary, draining an area of 109,000 square miles, enters the Columbia River in Washington just north of Oregon (see plate I). Important tributaries north of the Snake include Clark Fork, which drains a large area in western Montana and enters the Columbia near the International Boundary; the Kootenai, which lies largely in Canada and enters the Columbia from the northeast 30 miles above the International Boundary; and the Spokane, Okanogan, Chelan, and Yakima Rivers. Important tributaries below the Snake are Walla Walla River, located in Oregon and Washington; the Umatilla, John Day, Deschutes, and Willamette Rivers in Oregon; and the Klickitat, White Salmon, Lewis, and Cowlitz Rivers in Washington.

3. Characteristics. - The following table locates various points of interest on the Columbia by river mileages from the mouth.

TABLE 1

Mileage from Mouth of Columbia River

| Miles | Point | Miles | Point |
|---|---|---|---|
| 14 | Astoria, Ore. | 292 | McNary Dam |
| 66 | Longview, Wash. | 312 | Port Kelley, Wash. |
| 75 | Kalama, Wash. | 314 | Wallula, Wash. |
| 101.5 | Mouth of Willamette River | 324 | Mouth of Snake River |
| 106 | Vancouver, Wash. | 328 | Pasco-Kennewick, Wash. |
| 145 | Bonneville Dam | 340 | Ben Franklin Dam Site |
| 149 | Cascade Locks, Ore. | 397 | Priest Rapids Dam |
| 189.5 | The Dalles, Ore. | 415 | Wanapum Dam Site |
| 192.5 | The Dalles Dam | 453 | Rock Island Dam |
| 201 | Celilo, Ore. | 465 | Wenatchee, Wash. |
| 217 | John Day Dam Site | 549 | Chief Joseph Dam |
| 242 | Arlington, Ore. | 599 | Grand Coulee Dam, Wash. |
| 289 | Umatilla, Ore. | 749 | International Boundary |

4. From the mouth to river mile 145, the river is characterized by many sand bars, islands, and channels of varying width and depth. This reach is affected by tidal action, diminishing from the mouth to Bonneville Dam. Velocities here are relatively low and the water-surface slope fairly uniform, averaging 0.056 feet per mile at low water.

5. Between miles 145 and 150, the Columbia breaks through the Cascade

THE COLUMBIA RIVER
PLATE I

Range in a spectacular gorge having a width from rim to rim of 1 to 2 miles. Basaltic cliffs rise to heights of more than 3,000 feet above the river. Below river mile 180, the terrain is heavily wooded, due to the predominately wet climate. Upstream to Canada the area is semi-arid and the river flows through sagebrush country.

6. Geology. - Geologically, the Cascade Range, oriented north and south, once formed a natural barrier to drainage from the east, and a large lake occupied the present basin. Evidence of the grey-sandstone lakebed and fossil remnants of shellfish testify to this today. The river cut through hundreds of feet of basalt in forming its present channel.

7. Climate. - The climate of the Columbia River Basin varies between mild coastal conditions at the river mouth to the semi-arid weather of inland valleys and plains, and to extreme temperatures found in the mountain altitudes. Precipitation is high in the western portion and low in the major, inland, eastern area. Most interior precipitation falls as snow during late fall and winter, being retained on high elevations until spring runoff. Annual precipitation ranges from 7 inches near mile 328, to 150 inches near the mouth of the river where it flows through the Coast Range. Droughts in the inland valleys are not uncommon and adversely affect crop production.

8. Hydrology. - Storms do not materially affect the main stem of the Columbia. Local winter storms cause flood conditions on the tributaries but usually have little effect on the flow in the Columbia. The following notable features of the runoff of the basin should be mentioned: 40.6 percent of runoff comes from the area in Canada and tributaries in the United States which join the Columbia in Canada; 36.1 percent comes from the area between the Cascade Range and the International Boundary, including the Snake River drainage; and 23.3 percent comes from the area west of the Cascade Range.

9. The largest known general flood in the Columbia River Basin occurred in June 1894. It resulted from rapid melting of an abnormal snowpack accumulated throughout the basin during the preceding winter. Maximum discharge was estimated at 680,000 c.f.s. at the International Boundary (mile 749) and 1,240,000 c.f.s. at The Dalles (mile 189).

10. Average annual high flows during the spring runoff are approximately 600,000 c.f.s., and normal minimum flows are about 100,000 c.f.s.

## B. History

1. As early as the 16th Century, Spanish explorers sailed past the mouth of the Columbia River without recognizing it. The white man's knowledge of The River of the West remained legendary to white men until its entrance May 11, 1792, by Captain Robert Gray in the American ship COLUMBIA, for which the river was named. Earlier navigators had suspected but failed to locate a large fresh-water stream in these latitudes, due probably to the sandbars which effectively screened its entrance. Further, unusually heavy breakers on the sandspits presented formidable difficulties to approach or passage by small sailing vessels in use at that time.

2. Captain Gray's ship was soon followed into the Columbia River by the American brig JENNY commanded by Captain Baker, whose name is borne by the bay just inside the entrance.

3. Captain George Vancouver, of the British Navy, commanding the sloop of war DISCOVERY and the armed tender CHATHAM, was conducting explorations in the north Pacific Ocean at the time of Captain Gray's discovery. Learning of the event, Captain Vancouver assigned further Columbia River exploration to Lt. W. R. Broughton who sailed a small boat to Vancouver Point, present site of Vancouver, Washington, in the fall of 1792.

4. By 1796, numerous ships in the fur trade were making the estuary of the Columbia a port of call.

5. Lewis and Clark, in 1805, reached the Snake River and traveled down to its junction with the Columbia. The descent to the mouth of the Columbia was made in canoes, with portages over the numerous rapids. The Lewis and Clark journal is the first authenticated instance of navigation by white men on the Columbia River above Vancouver.

6. The Columbia, because of its importance as an avenue to inland areas, has been a factor in many historic events.

## C. Early Navigation

1. Fur traders, seeking sea otter and other pelts for the China trade, pioneered the first routes inland from the mouth of the Columbia during the early years following its discovery. Ships, longboats, canoes, and bateaus — rafts propelled by pole and sail — were used by these earliest white travelers on the river.

2. Astoria, named for John Jacob Astor, was founded on the Oregon shore in 1811 as an American fur trading post. But a scant 2 years later, war and failures of supply forced sale to the British Northwest Company, and American trading waned on the river. Merger of the Northwest Company and the Hudson Bay Company in 1821 consolidated English predominance in the trade for several decades to follow.

3. A calculated policy of saturation trapping, to maximize profit and discourage permanent settlement, sent trapping parties, by canoe and bateau, into the upper reaches of the Columbia and major tributaries. For the lower river system, company plans were for permanent occupation and trading with Indian trappers on a scale planned to conserve the fur resource.

4. By 1828, Fort Vancouver was firmly established as operating headquarters for the Columbia Department of the company. Other posts were Flathead House, Spokane House, Fort Nez Perce (Walla Walla), and Fort George (Astoria). Under the over-all direction of the company's George Simpson, Chief Factor John McLoughlin directed the flow by river and pack train of supplies to and furs from these outposts. English shipping supplied Vancouver and delivered the furs into British channels of trade. Little other

traffic moved on the Columbia River until the 1840's.

5. The English-built BEAVER, owned by the Hudson Bay Company, was the first steamship to enter and ply Columbia River waters. For nearly 50 years she operated in the northwest until she was wrecked in the Canadian harbor of Vancouver in 1883. The first substantial craft built on the Columbia was a two-mastered schooner of about 85 tons launched at and named for Vancouver.

6. In the early 1840's, dominance of Hudson Bay Company was challenged. With the arrival of missionaries, Protestant and Catholic, and the coming of American, English, and French settlers, increasing areas were cleared, and trappers, perforce, moved repeatedly deeper into the forested wilderness. Lumbering and fisheries developed rapidly, and small sailing vessels carried their products to gold-rushed San Francisco and the Pacific islands. Soon, small sailing vessels plied regularly between the Columbia and Willamette River settlements at Astoria, Vancouver, Portland, and Oregon City.

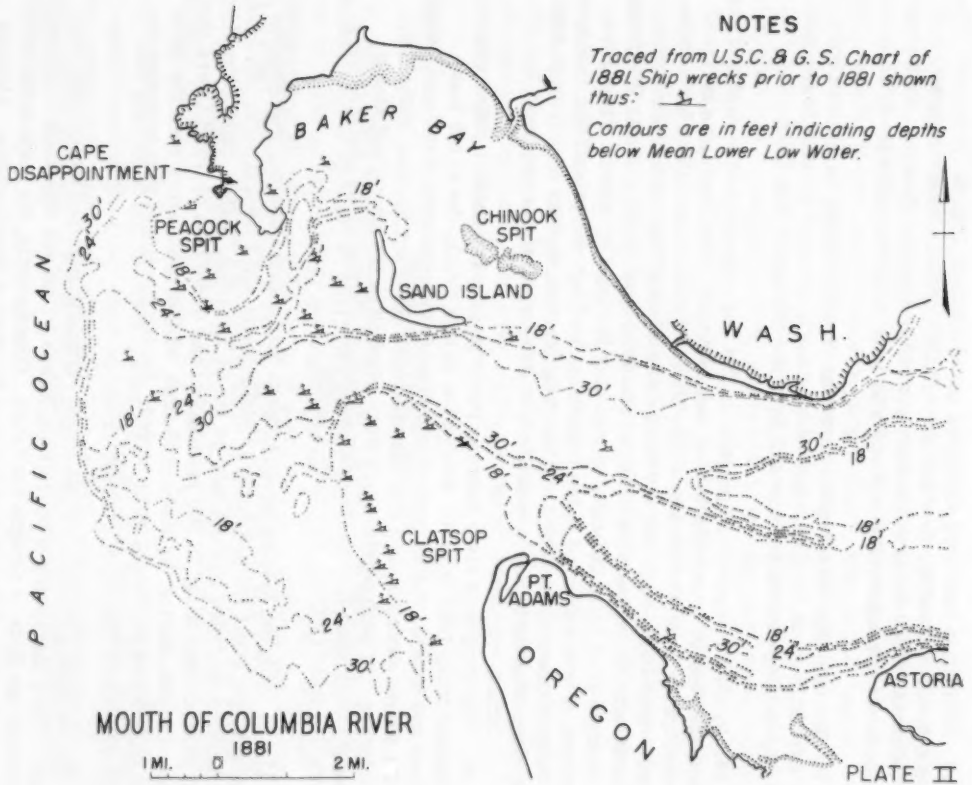
7. Ship movement up the river and across the treacherous entrance bar was not always easy nor safe. In 1841 the U. S. Sloop PEACOCK was wrecked on the spit which now bears her name. Plate No. II shows the sites where some of a reported 2,000 ships have been lost at or near the mouth of the Columbia. The Cape Disappointment lighthouse was built in 1856 to assist navigators.

8. In 1842, Captain Couch sailed his brig, the CHENAMUS, up the Columbia and Willamette Rivers to Clackamas Rapids, just below Oregon City. The captain established a store at what is now Portland, Oregon. Although Oregon City and Milwaukie were settled before Portland, their growth was impeded by difficulty in navigating the Willamette above Portland. If improvements for navigation had been available then, Oregon City, rather than Portland, might well have been the largest city in Oregon. The growing importance of marine traffic to the region was recognized by the Oregon Territorial legislature, and a pilot commission was appointed in 1846.

9. Movement of produce by water, the only means available, was difficult and slow. Sailing craft required from 10 to 50 days to negotiate the 112 miles between Portland and the sea. It was not until about 1850, when steam vessels appeared, that a marked increase in volume of waterborne traffic occurred.

10. The first steamship built in the northwest was the COLUMBIA, a side wheeler 90 feet long and drawing 4 feet. She was launched near Astoria on July 3, 1850, proceeded immediately upstream, and reached Portland on the 4th of July. She controlled business for only 6 months until the steamer LOT WHITCOMB was launched at Milwaukie. This vessel was 160 feet long and had a draft of 5 feet 8 inches. She was well built and appointed to attract passengers. Soon, the WILLAMETTE, a propeller-driven ship, entered competition.

11. Portland made its bid in 1851 as a seaport of importance. It was then possible to travel on schedule either to San Francisco or up or down river.


NOTES
Traced from U.S.C. & G. S. Chart of 1881. Ship wrecks prior to 1881 shown thus:
Contours are in feet indicating depths below Mean Lower Low Water.
BAKER BAY
CAPE DISAPPOINTMENT
PEACOCK SPIT
CHINOOK SPIT
SAND ISLAND
WASH.
PACIFIC OCEAN
CLATSOP SPIT
PT. ADAMS
OREGON
ASTORIA
MOUTH OF COLUMBIA RIVER
1881
1 MI. 0 2 MI.
PLATE II

12. These were the exciting days of river navigation. Many scheduled or impromptu races developed. Each captain felt that he could command more business if he could boast of the fastest ship.

13. Navigation was first attempted above Cascade Rapids (river mile 149) when a small steamship, the JAMES P. FLINT, a 60-foot side wheeler, was placed on the river between Cascade and Celilo Rapids. However, sufficient business did not materialize and she was soon returned to the lower river.

14. Recognizing the potential above the rapids, a group of settlers built a portage railroad at the Cascades. Subsequently, several vessels began making regular trips to the portage and from the portage to The Dalles.

15. In 1856 a wagon-road portage at The Dalles permitted freight and passenger movement above The Dalles, and regularly scheduled trips became commonplace from Portland to Walla Walla. Freight was $100 a ton between these points. Such cost was understandable when all the difficulties were considered. An article to be shipped from the East Coast to Walla Walla had but comparative little difficulty in reaching Portland by water. From here, however, it was subjected to ten transfers, loadings and unloadings, transport on three steamers, one short and one long portage, and the long freight-wagon haul from Wallula to Walla Walla before delivery to the consignee.

16. The stretch of river between Pasco and the Okanogan country was navigated from 1861 to 1890. Navigation in this reach, due to the many rapids and shoals, was hazardous and required experienced river men. Many of the rapids were so swift that the steamers could not advance under their own power. At such points, lines were secured to the shore, and steam winches were used to inch the craft upstream. A few hardy pioneers even propelled their craft up river into Canada.

17. In 1861, the famous Oregon Steam Navigation Company was founded. It served the river from Astoria to Walla Walla and Priest Rapids. By 1862 its service extended up the Snake as far as Lewiston, Idaho. During the first full year of operation, 10,500 passengers and 6,290 tons of freight were carried. In 1864, during the Idaho gold rush, 36,000 passengers rode the boats and 21,834 tons of freight were moved. During the life of this company, its fleet numbered over 70 steamships.

18. In 1863, privately-owned steam railroads assumed the portage burdens. Their monopolistic operations, however, caused dependent boatmen to approach the State of Oregon for relief. Consequently, the State underwrote the construction of the Oregon Portage Railway at the Cascades. Although business was good it was not profitable, yet it broke the monopoly and served until 1896 when Cascade Locks were completed.

19. A similar situation occurred at The Dalles, and the State again assisted in construction of a portage railroad called the Oregon State Portage Railroad. It too broke the monopoly and continued until completion of Celilo locks and canal in 1915.

20. First shipments of lumber from the Portland and Milwaukee area were in 1853 and proved to be a stimulus to this trade, which even today is a mainstay of the port's economy. Another first was the movement of 200 barrels of flour to the Orient in 1855.

21. During the 1860's, overseas shipping suffered since river conditions prevented the larger ships from operating economically between the mouth and Portland. Lumber and other products for overseas shipment were lightered to St. Helens and reloaded on larger ships. Also, small coastal craft transported products to San Francisco for transshipment overseas.

22. Because of resultant high freight costs and consequent business stagnation, Portland residents, in 1864, donated funds for shipping improvements.

23. With increased use of the Columbia for navigation, came need for larger and better channels. Therefore, in 1871, the Federal Government established the Portland District office of the Corps of Engineers. Previously, Government activities had been conducted by the San Francisco District office.

24. As the river was gradually improved for navigation, the amount of traffic and passengers generally increased. However, when railroads and roads appeared, the volume of river traffic declined. Passenger traffic shifted promptly to faster land transport.

## D. Improvements for Navigation

1. Just prior to the beginning of improvements on the Columbia, a book was published in London, in 1853, entitled "Sailing Directions for the West Coast of North America." Its description of the Columbia says: "Although it possess's at all times a good depth of water, yet it is difficult and dangerous to enter so much so that it can be said to possess but few advantages as a port." Change from that unprepossessing beginning to its current status as one of the important navigable rivers of the world has required a long, hard, expensive effort.

2. This portion of the discussion conforms substantially to the chronological improvement of the river when obstructions were removed, dikes and wing dams constructed, dredging performed, jetties built, and channels, locks, and multi-purpose projects including provisions for navigation were planned and constructed.

3. Snagging and rock removal. - Removal of snags was the first recorded improvement of the river. As the river channels became more stabilized through annual dredging activities, need for snagging has diminished. Large, powerful dredges currently in use either break up the snags into smaller pieces, which carry through with the dredged materials, or disposes of them by burial or placement on shore, as incidental to the dredging operations. Snags deposited by high water in the shallow sections of the river are a hazard to the drift nets of fishermen and are quickly removed.

4. First rock removed from the upper Columbia River was at John Day Rapids in 1873. Incidently, these rapids will be the last on the upper Columbia to be inundated when John Day Dam is built.

5. Rock reefs were later removed at Umatilla Rapids and Homly Rapids, near the mouth of Snake River. Removal was accomplished by drilling from a barge and blasting; a procedure very little different from that of today. Boulders not large enough to require blasting were either raked from the channel or into deep water.

6. Above mile 145 it proved more economical to construct locks and canals and, subsequently, locks and dams, than to carve an adequate channel out of the rocky river bed.

7. One of the most interesting early improvements for navigation was the work done in the upper Columbia River near Priest Rapids. Here, early steamers used shore lines to overcome the high velocities. To aid in this type of navigation, numerous eye bolts and rings were installed along the bank.

8. Pile dike construction. – Construction of pile and timber dikes, wing dams, and rockfilled cribs has been mostly confined to the lower 137 miles of Columbia River and Willamette River below Portland, Oregon. Because of high river velocities, solid rock reefs, and bad icing conditions, such works are unsatisfactory on the upper Columbia River.

9. The lower Willamette and Columbia Rivers are braided streams with islands which divide the flow. As increased channel depths were demanded, maintenance with bucket dredges became an increasing problem, and it was decided to construct dikes and dams across auxiliary channels in an attempt to increase self-scouring velocities along the main ship channel.

10. The first work of this type was initiated in 1875 with construction of a 900-foot, pile-and-timber dam across an overflow channel in the Willamette River located about 2 miles upstream from the mouth.

11. During the spring of 1876, a freshet carried away much of the structure. This failure was attributed to powerful river currents, a heavy drift accumulation, inadequate pile penetration, a foundation consisting of quicksand, drift bolts of inadequate length and dimensions, and insufficient brush and stone placement to prevent scouring action by whirlpools and eddies.

12. In 1879, wing dams were constructed in Willamette Slough near the river mouth. These dams were constructed to project 2 feet above mean low water. Two solid rows of piling, 10 feet apart, were driven out from each bank, leaving a gap about 150 feet wide near the center of the waterway as a boat channel. Construction also began on a substantially similar dam extending from the head of Sauvies Island to Coon Island, a distance of 1,577 feet. During 1883, a total of 12,821 linear feet of pile dikes, wing dams, and bank revetments were built along the lower Willamette River. These installations are still effective in raising the low-water plane of Willamette River from 1 to 1-1/2 feet and have caused shoaling of secondary channels until now only

one waterway exists in the Willamette River below Portland.

13. Effectiveness of these early dikes and dams on the lower Willamette River indicated that dredging could be reduced on many Columbia River bars; accordingly, pile dike construction was extended to include the more critical areas on the lower Columbia. Methods of construction and types of closure structures remained relatively unchanged until the River and Harbor Act of July 25, 1912, specified a channel 30 feet deep, 300 feet wide from Portland to the sea, to be accomplished by dikes and revetments as supplements to dredging. This increase in project dimensions required dredging on many bars where depths had been adequate previously. Major shoaling takes place during the spring freshet, which usually occurs between May 1 and July 15. But, during these freshets, high river velocites and increased depths prohibit dredging. This results in difficulties in maintaining full channel dimensions at all times. When the river drops, most bars require immediate dredging to remove shoals, and with the few dredges available this is impossible. Therefore, a system of dikes was planned and constructed to extend channelward from the shoreline attempting to reduce velocities alongshore and increase velocities in the desired channel.

14. The solid, dam-like structures used earlier to close auxiliary channels could not be used now as such installations constructed on sand foundations develop an upstream eddy alongshore, and water passing over the structures and along their outer ends become so turbulent that resultant scour undermines the structures. Therefore, an entirely new type of permeable dike, proven satisfactory on other waterways, was used. In recent years, minor changes have been made in construction details but basically the design is unchanged; two rows of piles spaced at 5-foot centers in each row, separated by a horizontal pile or timber. One row of pile is 2.5 feet in advance of the other to provide a pile every 2.5 feet along the dike. It was necessary to place revetment stone at the base of the pile for the full length of the dike, around the outer end of the dike, and alongshore upstream and downstream from the dike to prevent undermining and outflanking.

15. A dike so constructed reduces velocity along its length and increases velocities channelward of the dike without creating excessive turbulence of a solid structure. The length and distance between dikes is determined by judgment and experience depending upon the width of the waterway and its changes in direction.

16. The life of an untreated pile dike in fresh water is unknown, as the portion which is submerged daily remains solid. Dikes originally constructed with a cutoff elevation 10 or 12 feet above low water are gradually lowered as top rot extends down the pile and they become loose at the point of contact with the spreader pile. As the more effective portion of the dike, from a channel maintenance standpoint, is the portion below low water, it will eventually be necessary to cut the dikes down to a point below which rot does not occur, and the bolting of the vertical pile to the horizontal spreader pile will require underwater work.

17. On the lower Columbia River, where all dikes are submerged during spring freshets, it has been found necessary because of the increased use of

the waterway by small, sport-fishing and recreational craft to place dike-marking dolphins, in conjunction with dike repairs, at 150-foot intervals along the length of the dikes.

18. There are a total of 187 dikes between the mouth and mile 145. Plate No. VII shows the construction details of a typical pile dike and how pile dikes have been located along the main navigation channel.

19. Jetties. - Between 1792 and 1885 there were from one to three channels across the entrance bar, varying frequently both in location and depth. Depths were from 19 to 21 feet at low water, and locations shifted through nearly 180 degrees between Cape Disappointment and Point Adams. The ocean beach and bed in the river estuary, and the spits, bars, and shoals, both within and without the entrance, are of a fine sand which is readily shifted by water and wind movement. Various futile attempts were made to improve conditions. The Corps of Engineers, in 1879, attempted unsuccessfully to harrow the channel during strong ebb tides. Propeller sluicing was tried on some bars. Such measures were, of course, temporary and the effects lost during the next freshet or storm. Surveys of the mouth have been made frequently since 1792. Plate II shows the condition of the bar during times prior to jetty construction.

20. The first project for improvement of the mouth, adopted in 1884, provided for construction of a low-tide jetty, 4-1/2 miles in length, extending seaward from Point Adams on the south side of the entrance and terminating about 3 miles south of Cape Disappointment. It was to be built of rubblestone with a foundation of brush mattresses. The design criteria for spacing the jetties was determined by examination of the estuary condition. The section between Chinook Point and Point Adams appeared to induce sufficient tidal and river action to maintain adequate channel depths. Therefore, the south jetty was located to provide a similar waterway between Cape Disappointment and Point Adams. The scope of the project was revised in 1893 to raise the jetty to high-tide level, and build four groins from 500 to 1,000 feet long on the north side. The jetty was completed to a length of 4-1/2 miles in 1895, and depth over the bar increased to 31 feet. By 1899 depth shallowed to 28 feet and the channel shifted 2 miles north of the 1895 channel. By 1902 the channel had deteriorated to depths of 21 feet at low water. In 1903 the project for the south jetty was restudied and modified to provide extension of the jetty seaward a distance of approximately 2-1/2 miles. Construction of a north jetty was also recommended, should it be needed to secure the desired depth of 40 feet. The section of the jetty was increased to a top width of not less than 25 feet at midtide elevation. During construction of the earlier jetty, storms took a considerable toll in destruction of pile trestles and equipment.

21. Over 5,800,000 tons of rock went into the south jetty, which was completed in 1913. Its total length was 6.62 miles. Most of the rock was quarried above Vancouver, Washington, near river mile 110, and barged downstream to site, placed on dump cars, and hauled by locomotive to the structure. Pile trestles were driven ahead of jetty construction, and the rock was dumped directly into place.

22. Plant and equipment from the south jetty was salvaged and moved to Fort Canby for use in construction of the north jetty, on which work started in February 1914. The north jetty, 2.35 miles long, completed in 1917, was constructed in the same manner as the south jetty and contained 4,000,000 tons of stone.

23. At the end of 1917, the channel was 41 feet deep. Between 1931 and 1937, the south jetty was repaired; 2,200,000 tons of rock being used. The outer end of the north jetty was repaired in 1938 and 1939 when 250,000 tons of rock were used.

24. Both the north and south jetties were modified by addition of concrete terminal sections; the north jetty in 1939, and the south jetty in 1941. The terminal on the south jetty was placed approximately 4,000 feet from the outer end and has been effective in stabilizing the terminal.

25. As a further aid in maintaining the channel along Peacock Spit on the north side of the estuary, a third inner jetty, Jetty "A," was constructed in 1939. It originally contained 240,000 tons of rock and was 10,000 feet long. It has since been repaired four times, utilizing about 25,000 tons of rock each time. Plate 3 shows locations of the jetties.

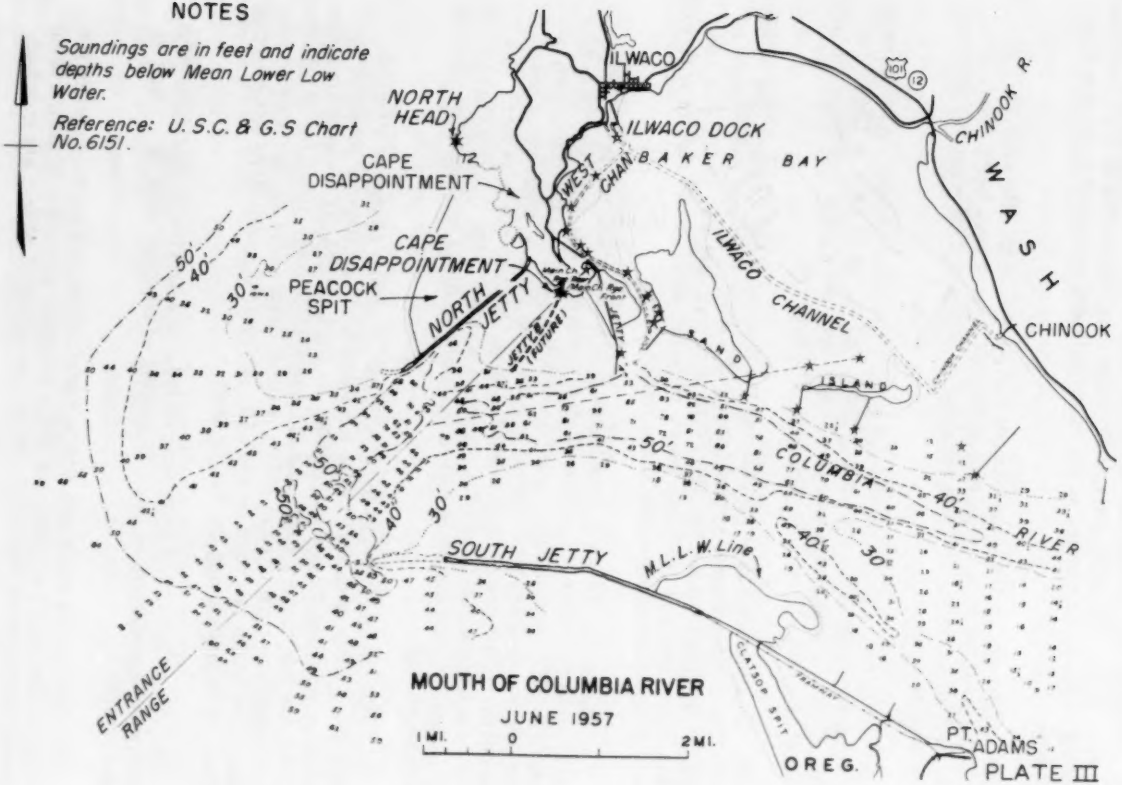
26. At present the south jetty has two subsided areas which will soon require repair to insure its continued effectiveness. The outer end is in fair condition. The north jetty is in good condition except at the connection between the outer concrete terminal and the jetty proper where subsidence has occurred and new rock is needed. Jetty "A" also needs repair. The outer end has subsided due to the erosive action of high velocity currents on the foundation, and the inner end is in danger of being outflanked by erosion of Peacock Spit.

27. The channel between the jetties has been maintained to a 40-foot depth for a width of one-half mile for many years by natural scouring assisted by occasional hopper dredging. In 1956, due to the demand for deeper drafts, the bar channel depth was increased to 48 feet. Dredging of this deeper channel was completed in 1957. A fourth jetty (jetty B), the approximate location of which is shown on plate III, has been authorized for construction if needed in maintaining the bar channel.

28. All jetties are of rubblemound type, constructed of maximum-size quarry rock available; at least 50 percent of the rock must be 10 tons or over in size.

29. Due to high winds and waves, construction and maintenance of the jetties at the mouth of Columbia River probably present difficulties unsurpassed elsewhere in the world. Waves in excess of 30 feet are common.

30. Good success has been experienced in maintaining other jetties by utilizing heavy, automotive hauling equipment operating on the crest. It is probable that future maintenance work on the Columbia River jetties will be done in this way.


NOTES
Soundings are in feet and indicate depths below Mean Lower Low Water.
Reference: U. S. C. & G. S Chart No. 6151.
ILWACO
NORTH HEAD
ILWACO DOCK
CAPE DISAPPOINTMENT
CAPE DISAPPOINTMENT
PEACOCK SPIT
NORTH JETTY
JETTY (FUTURE)
WEST CHANNEL
BAKER BAY
ILWACO CHANNEL
SAND ISLAND
CHINOOK R
WASH
CHINOOK
COLUMBIA RIVER
SOUTH JETTY
M.L.L.W. Line
ENTRANCE RANGE
GLATSOP SPIT
PT. ADAMS
OREG.
MOUTH OF COLUMBIA RIVER
JUNE 1957
1 MI. 0 2 MI.
PLATE III

31. Dredging. - Maintenance of a ship channel in the lower Willamette River was begun by the city of Portland in 1865, using a bucket dredge and barges for shoal removal. Shortly thereafter the United States, under the River and Harbor Act of June 23, 1866, began to participate in river maintenance. Actual work did not begin until 1868, when the Corps of Engineers agreed to use and maintain a city dredge and barges in the river without cost to the city. First Government work was dredging a channel 900 feet long, 100 feet wide, and 15 feet deep through Swan Island Bar in Portland Harbor.

32. During 1873 the Government's responsibility extended to include improvement of the lower Willamette and Columbia Rivers from Portland to the sea. After 5-1/2 years of operation, the Corps returned the city's dredge and constructed a 1-1/2-cubic-yard bucket dredge with barges for their own use.

33. The spring freshet of 1883 shoaled the bar at St. Helens to 12.5 feet, and Federal funds were not available for removal. The work was undertaken by private interests, using the iron steamship WALLA WALLA which was secured to the banks in such a manner that it could be maneuvered from side to side and, with the aid of its powerful propellers, sluiced the material off the bar into deeper water, providing a depth of 22 feet. This method became accepted practice for several years, and a private firm equipped a large tugboat for this job.

34. The city of Portland built an endless-chain bucket dredge for use in the lower Willamette River. Operation was limited to depths of less than 18 feet, and it was damaged whenever hard digging was encountered.

35. In 1891 the port of Portland, a municipal corporation, was organized to provide and maintain a ship channel of at least 25 feet in depth in the Willamette and Columbia Rivers between Portland and the Pacific Ocean; the work to be coordinated with that performed by the Government. The River and Harbor Act of 1892 officially provided a 25-foot channel.

36. In 1893 the Corps secured a hopper dredge WILLIAM S. LADD, having a capacity of 300 cubic yards. By September 1894 the channel to the sea had been sufficiently improved to permit the passage of fully loaded ships from Portland.

37. It proved impossible to maintain the 25-foot depth with the equipment of the port of Portland and the Corps of Engineers. Consequently, the port bought a 20-inch suction dredge, and later, in 1903, a 30-inch dredge was added.

38. Maintenance dredging and building additional pile dikes gradually produced a deeper and deeper channel. Usual procedure was to use the two large pipeline dredges in the upper reaches and the U. S. Government hopper dredge in the estuary. The pipeline-dredge operation was financed by the Government when funds were available, otherwise by the port of Portland. Eventually the channel became less expensive to maintain as the river adjusted to the new section.

39. In 1912 a channel 30 feet deep and 300 feet wide from Portland to Brookfield, and not less than 26 feet deep from there to the sea, was authorized. To do this additional work, the port of Portland obtained a second 30-inch dredge, and the Corps of Engineers provided two 24-inch dredges, the MULTNOMAH and WAHKIAKUM. These Government dredges are still operating.

40. In 1915, the first year that all this equipment was available, over 11 million cubic yards of material were moved, using two hopper dredges and two 24-inch pipeline dredges owned by the Government, and the 20-inch and two 30-inch dredges of the port.

41. The channel between the mouth of Williamette River and Vancouver, Washington, was first authorized in 1916. It has been repeatedly increased in dimensions until it is now 30 feet by 300 feet.

42. In 1930 the channel project between Portland and the sea was altered to provide a depth of 35 feet and a width of 500 feet, its present size. The Government has increasingly assumed maintenance of the project until now the port of Portland only maintains a 30-inch dredge for Government use on a cost-reimbursable basis. Each port district is responsible for dredging required within 50 feet of the pierhead line.

43. Since 1930 there has been little change in dredging methods. Most shoaling occurs during spring freshets. Frequent periodic hydrographic surveys show that the bed sand moves downstream in waves during high water. Due to this movement of drift, and increased depths, no dredging is done until the freshets subside. As soon as practicable after a freshet, hydrographic surveys are made of sites and degrees of shoaling. The dredges are then started on most critical areas and continue work until the entire channel is restored to proper depth. Dredges normally excavate 2 or more feet below project depth to allow for inaccuracies and to provide minimum project depths for a longer period.

44. There are 28 bars on the lower Columbia River, averaging approximately 2 miles in length, requiring annual maintenance.

45. Because of freshet and weather conditions, the dredging season is limited to about 6 months of the year. Pipeline dredges require quiet water to prevent damage to equipment and floating discharge lines; therefore, dredging at the mouth and in the estuary, which is subject to swells, is performed by hopper dredges.

46. Recently, removal of approximately 10 million yards annually from Columbia River between Portland and Vancouver and the mouth has been necessary, and last year approximately 4,000,000 cubic yards were dredged from the bar channel at the mouth. Dredging of the outer bar has been done by the hopper dredge BIDDLE, which also works on the estuary bars. Upstream channels have been dredged by two Government 24-inch dredges, the MULTNOMAH and WAHKIAKUM, assisted by the port of Portland's 30-inch CLACKAMAS and contract dredging.

47. Spoil has been disposed of principally between pile dikes and along the bankline. Several river islands grew from years of accumulation of dredge spoils. Dredge disposal is placed on lands adjacent to the river when the owners pay for the additional cost of handling the shore-pipe discharge lines.

48. The disposal-area problem along the entire Columbia River between Bonneville and the mouth is becoming increasingly acute as available disposal areas are filled, and longer pipelines and booster barges are required. Disposal of dredge spoils in the port of Portland area is now critical. Only two disposal areas remain in this 10-mile stretch and these cannot be reached by pipeline dredges without booster pumps and shoreline equipment.

49. Locks and channels. - As navigation demands increased in the upper Columbia River, the two railroad portages at the Cascades, mile 141, and Celilo, mile 201, became increasingly inadequate. The Corps of Engineers made surveys of the sites in 1874 and recommended channels and locks at both places.

a. Cascade Canal and Locks. - The Cascades were a series of rapids 5-1/2 miles in length. At the head of the rapids, a rocky bar extended entirely across the river closing the channel completely and, at low-water, caused a fall of 24 feet in a distance of 2,500 feet. Congress authorized the project soon after the recommendation was made; initial funds for the work were appropriated in 1876. The project formally opened for navigation in 1896. The canal provided a depth of 8 feet at low water, for a distance of 3,000 feet. The two locks were 521 and 514 feet long and 90 feet wide. Before the canal was built, the average annual freight over the portage road between 1892 and 1895 was 8,000 tons, and 7,500 passengers were carried. In 1906, ten years later, traffic through the channel amounted to 46,884 tons of freight and 133,070 passengers.

b. Celilo Canal and Locks. - This improvement was necessary to overcome the obstructions to navigation in the 10 miles of Columbia River between the foot of Threemile Rapids, mile 191, and the head of Celilo Falls, river mile 201. Between these points the fall of the river is about 81 feet at low water and 60 feet at high stages.

c. The original project adopted by Congress was to provide a boat railway. Land was purchased in 1900, one survey was made, and the project was revised to provide a canal and locks. The facility built included a canal on the Oregon shore from Celilo Falls to Big Eddy, a distance of 8-1/2 miles. The canal was 65 feet wide, 8 feet deep, with five locks 265 feet long, 45 to 50 feet wide, and 7 feet deep.

d. The one steamboat company operating on Columbia River above Celilo, Oregon, before opening of the canal, suspended operations in 1912 owing to lack of patronage. From then until the opening of the canal in 1915 no river freight was transferred around the obstructions in the river between The Dalles and Celilo, Oregon. Soon after the opening of the canal, a through service from Portland, Oregon, to Lewiston, Idaho, was established.

e. Bonneville Dam. - Bonneville Dam, a multiple-purpose project, was authorized in 1933 by the Federal Emergency Administration of Public Works. Authorization provided for a dam, powerplant, and navigation lock. The lock,

opened to traffic in January 1938, has a single lift with clear dimensions of 76 feet in width, 500 feet in length, and a depth of 24.2 feet over the sills at low water, with a lift of 66 feet at extreme low water and normal pool level.

f. The Cascade Canal and Locks were inundated by the pool at Bonneville in February 1938. Bonneville Pool extends to above The Dalles, Oregon, providing slackwater navigation for 40 miles upstream, with depths of 27 feet or more (See plate No. IV for general plan of Bonneville Dam and location of the abandoned Cascade Locks.)

g. The Dalles Dam. - The Dalles Dam is also a multiple-purpose project for power and navigation. It was authorized in 1950, construction started in 1951, and the project is approximately 90 percent complete. The pool, raised in 1957, creates a reservoir upstream 23 miles to the John Day dam site. The project consists of a spillway, powerhouse, and navigation lock. The lock is 86 feet wide and 675 feet long, with a normal depth over the upper sill of 20 feet and a lift of 89.5 feet. It was opened to navigation in March 1957, completing another increment of slackwater and eliminating The Dalles-Celilo Canal. The pool provides a slackwater channel with depths of 20 feet and over from the dam to the site of future John Day Dam. Plate No. IV indicates the general plan of the dam and lock.

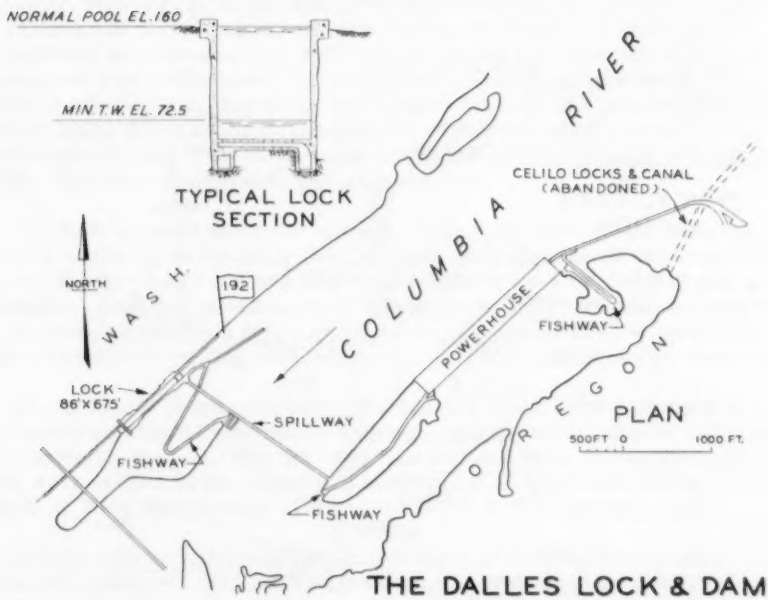
h. McNary Dam. - McNary Dam, at river mile 292, was authorized in 1945, construction started in 1947, and the project completed in 1957. It includes a dam, powerhouse, and navigation lock. The lock is 86 feet wide, 675 feet long, and has a normal lift of 84.4 feet. Depth over the upper sill is 20 feet at normal pool elevation. A plan of the project is shown on plate V. This project provides slackwater navigation up the Columbia approximately 64 miles to a point about 27 miles above Pasco and Kennewick, Washington. The pool created by the dam provides a navigation channel with depths of 12 feet and over between McNary and the confluence of the Yakima River. The pool also extends up the Snake River to the foot of Ice Harbor Dam and has inundated the Umatilla and Homly Rapids which were once a hazard to navigation.

i. Ice Harbor Dam. - Congress in 1945 authorized the construction of a system of four multipurpose dams in the lower Snake River from its confluence with the Columbia to Lewiston, Idaho, to provide slackwater navigation and power.

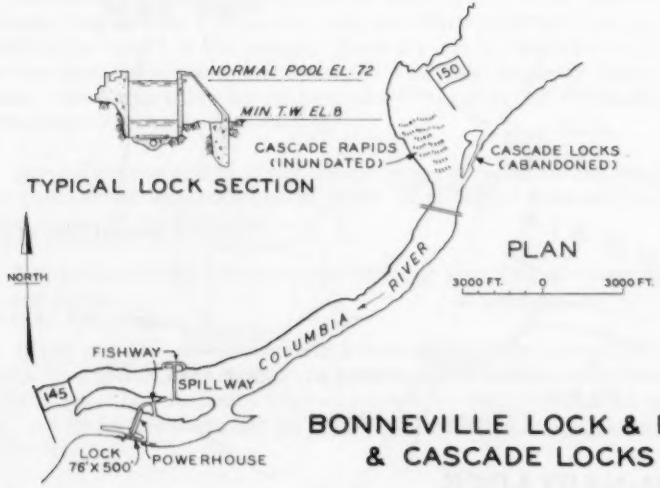
j. The present controlling depth of the channel for this reach, a distance of 140 miles, is less than 1 foot. Ice Harbor, first of the four dams, is located 10.4 miles above the mouth of the Snake at the upper end of McNary Pool. Construction of this dam started in January 1956. It will have a navigation lock 86 feet wide and 675 feet long. The dam will create a slackwater pool upstream 34.5 miles to the base of the next dam in the series, Lower Monumental. Plate V shows the general arrangement of this project.

k. Priest Rapids Dam. - This dam is under construction in the Columbia River at mile 397 by a public utility district licensed by the Federal Power Commission. It is being constructed as a power dam with provision for a ship lock when need arises.

50. Hydrographic surveys. - Hydrographic, condition surveys are made at least twice a year of the entire stretch of Columbia River between the mouth and Bonneville Dam and in Willamette River to Corvallis. Periodic surveys



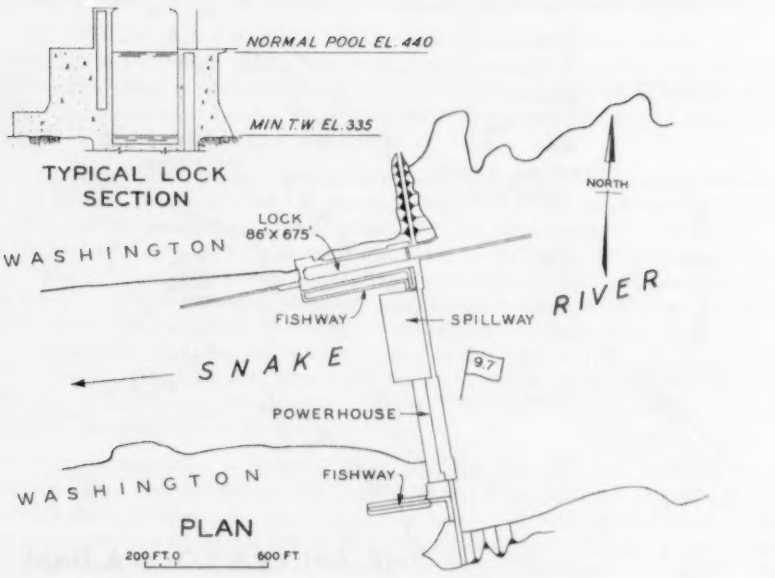

THE DALLES LOCK & DAM AND CELILO LOCKS



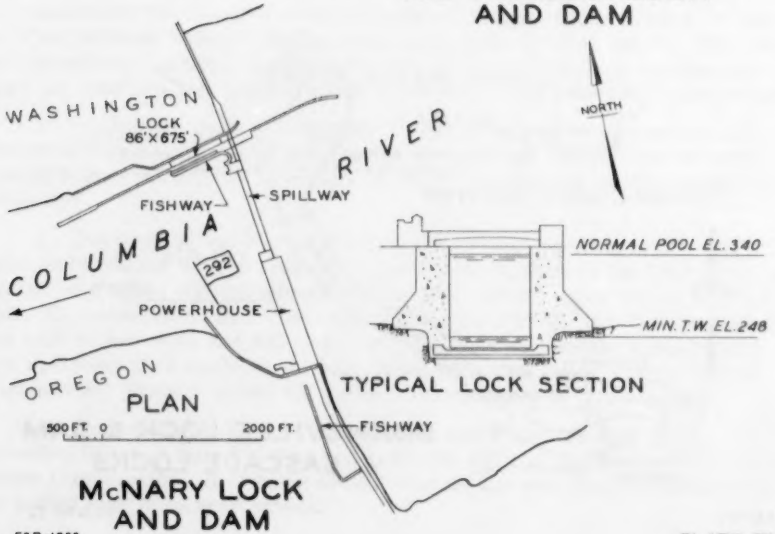

BONNEVILLE LOCK & DAM & CASCADE LOCKS

FGB-1958

PLATE IV




ICE HARBOR LOCK AND DAM




McNARY LOCK AND DAM

F.G.B.-1958

PLATE V

are made of side channels and tributaries as required. In areas requiring dredging, surveys are made before and after dredging to determine quantities to remove and the subsequent channel conditions. Copies of surveys are available to anyone concerned with channel depths and conditions. Captain Vancouver reportedly made the first hydrographic sketch of the Columbia River mouth in 1792. Soundings were taken with lead line and horizontal control obtained with a sextant. Considerable detail was shown in the first comprehensive survey of the estuary and ocean made by Sir Edward Belcher in 1839. The main channel then was in Baker Bay.

51. Hydrographic work was more thorough after steamboats were employed, beginning in the early 1850's. When sounding, the steamer moved at a rate of 3 to 4 knots and was kept on the desired lines by sextant fixes or by compass. Alternate soundings were taken by two leadsmen, one on each side of the boat, at quarter-, half-, or whole-minute intervals. The lead lines were tested each evening, and indicated corrections applied to the soundings.

52. In recent years, electronic transducers which record on a paper roll are employed almost universally for depth determination. Use of radio location systems, such as "Raydist" and "Loran," are being increasingly accepted. The work is done by the Corps of Engineers, U. S. Coast and Geodetic Survey, the U. S. Navy Hydrographic Office, and the U. S. Geological Survey.

53. Navigation aids. - Responsibility for providing and maintaining navigation aids rests with the U. S. Coast Guard. They mark channel alinement and limits by lights, buoys, and ranges.

54. Whenever possible, each course is marked by ranges, usually at one end. These ranges have a front and rear structure separated by approximately one-tenth of the length of the course. Each structure carries a large white marker for daytime use and is illuminated with high-intensity lights for night navigation. Navigators line up the two markers and proceed until the course is intersected by a succeeding range.

55. Around curves and in areas where range structures are not practical, regular colored and lighted buoys or lights on piling or dolphins are used to mark the limits of the channel.

56. Navigation charts are prepared showing locations of range markers, buoys, and lights.

57. River regulation. - In the plan for comprehensive development of Columbia River, sufficient storage is envisioned to reduce river flows from 1,240,000 c.f.s., the maximum flood of record, to 800,000 c.f.s. or less at The Dalles. All locks are designed for uninterrupted operation at flows up to 800,000 c.f.s.

58. In 1948 a flood of 1,025,000 c.f.s. occurred in Columbia River and prevented shipping in many areas. High velocities, debris, and unusual eddy conditions created hazards which practically halted navigation.

E. Present Development

1. Present condition. - The current Corps of Engineers river-channel project for the Columbia, Snake, and lower Willamette Rivers provides for a channel 500 feet wide and 35 feet deep from the 48-foot project at the mouth of Columbia River to the Broadway Bridge in Portland, and a 30-foot channel from the mouth of Willamette River to the Interstate highway bridge at Vancouver. From Vancouver to The Dalles the channel, authorized 27 feet deep and 300 feet wide, is maintained to a 15-foot depth. Above The Dalles the channel is generally 15 feet deep to Pasco and Kennewick, with adequate widths for safe tug and barge traffic. Due to limited depths above this point, both in the Snake and Columbia River, no practical navigation exists.

2. Rarely are full project dimensions available for an entire year, although intensive, timely dredging reduces seasonal shoaling to a minimum. Despite this problem, traffic moves on the river throughout most of each year relatively free of tidal delays and short-loading.

3. Commercial traffic. - In the 51 years, 1905 to 1956, the volume of seaborne tonnage moving into and out of the river increased more than tenfold from 947,000 tons to over 11 million tons.

4. Of the above tonnage for 1956, outbound shipments represented over 46 percent of the total, the two major components being farm products, largely wheat, first, followed by forest products, mostly lumber. Inbound cargoes, comprising a little less than 54 percent of the seaborne traffic for the year, were overwhelmingly made up of petroleum products. The growing volume of exports and imports quite closely reflects the economic and industrial growth of the tributary area during the half century. Internal commerce in 1956, which includes the rapidly growing barge traffic with the interior, was almost as heavy as deep-sea traffic.

5. Traffic on the river system above Vancouver consists exclusively, at present, of barge tows and rafted logs and poles. (See plate VIII, which graphically depicts growth of waterborns commerce.) From negligible amounts before the opening of the Bonneville Lock in 1938, such upper river traffic had grown to over 1,500,000 tons by 1956. By far the greater part of this tonnage, nearly 700,000 tons, was upbound petroleum products transferred from oceangoing bottoms for barge transport to inland distribution centers located mostly on McNary Dam pool near the confluence of the Snake and Columbia Rivers. Largest downstream components of this tonnage above Vancouver were 290,000 tons of wheat and other grains and 328,000 tons of rafted logs, which went to downstream processors. Such elements of upper river tonnage as originated from seaborne traffic or so moved out of the river is included in the tabulation of oceanborne commerce.

6. Vessel types. - The traffic originating outside the river system or destined for outside points was carried in vessels as shown in the following tabulation:

External traffic

| Item | Years | | | | |
|---|---|---|---|---|---|
| | 1905 | 1925 | 1940 | 1950 | 1955 |
| Number of ship passages | 1,391 | 3,372 | 3,202 | 3,270 | 3,288 |
| Average cargo (tons per ship) | 681 | 1,551 | 2,162 | 2,818 | 3,320 |
| Average net registered tons | 1,186 | 2,450 | 3,264 | 4,544 | 4,780 |
| Reported drafts: | | | | | |
| Under 28 feet | 1,391 | 3,302 | 3,106 | 2,467 | 2,549 |
| 28 to 30 feet | - | 66 | 89 | 405 | 271 |
| 30 feet and over | - | 4 | 7 | 398 | 468 |

SOURCE: Commercial Statistics, U. S. Army Corps of Engineers.

7. Deep-sea traffic entering and leaving the Columbia River over the past half century has changed significantly. During 1905, when 1,391 ships used the river, total tonnage carried was under 1 million tons, and average cargoes only 681 tons. Most of the ships calling then were lumber carriers of relatively-light registered tonnage and shallow draft. From 1925 through 1955 the number of ship passages remained fairly constant, but cargo carried increased from about 4 million tons to nearly 11 million. Indicative of progressive change in cargo patterns is the growing number of vessels plying the river at drafts greater than 28 feet. These represent, predominantly, petroleum tankers upbound and grain, coal, and scrap carriers downbound. The practical draft limit, using the present channel, approaches 32 feet, although when fully loaded many of the tankers would draw over 32 feet.

8. Many vessels reported as of less than 28 feet draft are operated on regular runs to the Atlantic Coast, Europe, or South American or Oriental countries by some one of the more than 50 dock-line companies serving Columbia River ports. Such ships call at several Pacific Coast ports for discharge and pick-up of partial cargoes and are rarely fully loaded upon arrival at or departure from Columbia River, which is normally an intermediate stop between calls at other Pacific Coast ports.

9. The Portland-Vancouver harbor is the usual point of transshipment to or from barges for deep-sea cargo shipped from or consigned to upstream ports. Such transshipments represent a considerable proportion of the water traffic moving above Vancouver.

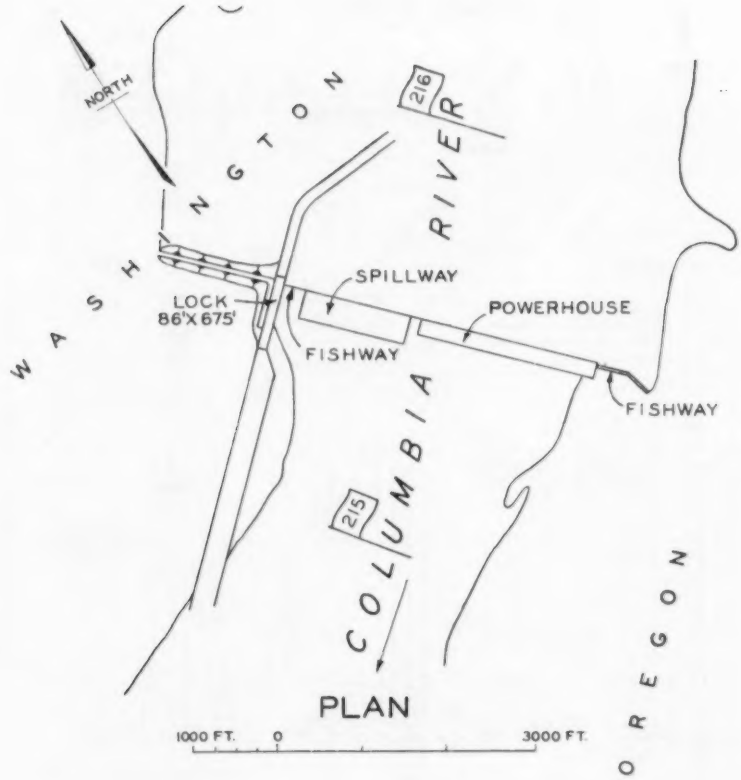
10. Most of this upper river traffic, aside from rafted logs, is moved by two barge lines. Equipment in use in this upper river service, includes some 70 barges and 35 tugs. Barges range in size from a minimum of 29 feet wide and 104 feet long, carrying 250 net tons, to a maximum of 48 feet wide, 260

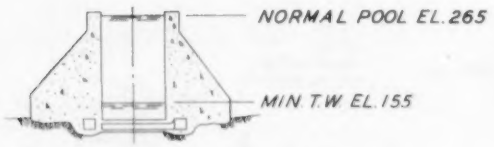
## TABLE 2

Commerce on Columbia River
By commodity groups, by 5-year averages
(in thousands of short tons)

| 5-year period | Products of agriculture | | Animals & products | | Products of mines | | Products of forests | | Petroleum & products | | Manufactures & misc. | | Total | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | M tons | Per-cent | M tons | Per-cent | M tons | Per-cent | M tons | Per-cent | M tons | Per-cent | M tons | Per-cent | M tons | Per-cent |
| Oceanborne commerce: | | | | | | | | | | | | | | |
| 1906-10 | 366 | 21.6 | 1 | 0.0 | 34 | 2.0 | 417 | 26.6 | 385 | 22.6 | 495 | 29.2 | 1,698 | 100.0 |
| 1911-15 | 596 | 22.1 | 3 | 0.1 | 19 | 0.7 | 864 | 32.0 | 785 | 29.1 | 432 | 16.0 | 2,699 | 100.0 |
| 1916-20 | 346 | 14.6 | 4 | 0.2 | 11 | 0.5 | 737 | 31.1 | 996 | 42.0 | 278 | 11.7 | 2,372 | 100.0 |
| 1921-25 | 1,093 | 23.7 | 52 | 1.1 | 31 | 0.7 | 1,524 | 33.1 | 1,392 | 30.2 | 515 | 11.2 | 4,607 | 100.0 |
| 1926-30 | 1,370 | 20.2 | 53 | 0.8 | 45 | 0.7 | 2,403 | 35.4 | 2,075 | 30.6 | 840 | 12.4 | 6,786 | 100.0 |
| 1931-35 | 982 | 17.4 | 51 | 0.9 | 63 | 1.1 | 1,629 | 28.9 | 2,150 | 38.1 | 762 | 13.5 | 5,637 | 100.0 |
| 1936-40 | 959 | 14.4 | 41 | 0.6 | 64 | 1.0 | 1,854 | 27.8 | 2,820 | 42.3 | 926 | 13.9 | 6,664 | 100.0 |
| 1941-45 | 270 | 4.2 | 136 | 2.1 | 66 | 1.0 | 778 | 12.0 | 4,279 | 66.2 | 934 | 14.5 | 6,463 | 100.0 |
| 1946-50 | 553 | 6.6 | 27 | 0.3 | 400 | 4.8 | 1,059 | 12.7 | 5,399 | 64.9 | 885 | 10.6 | 8,323 | 100.0 |
| 1951-55 | 2,150 | 21.0 | 46 | 0.4 | 483 | 4.7 | 1,168 | 11.4 | 6,073 | 59.4 | 307 | 3.0 | 10,227 | 100.0 |
| 1956 | 3,675 | 32.8 | 66 | 0.6 | 654 | 5.8 | 683 | 6.1 | 5,634 | 50.3 | 495 | 4.4 | 11,207 | 100.0 |
| Internal commerce: | | | | | | | | | | | | | | |
| 1951-55 | 388 | 4.8 | 17 | 0.2 | 784 | 9.7 | 5,824 | 72.3 | 916 | 11.4 | 128 | 1.6 | 8,057 | 100.0 |
| 1956 | 297 | 2.9 | 169 | 1.6 | 929 | 9.0 | 6,660 | 64.2 | 2,106 | 20.3 | 208 | 2.0 | 10,369 | 100.0 |

SOURCE: Commercial Statistics, U. S. Army Corps of Engineers.



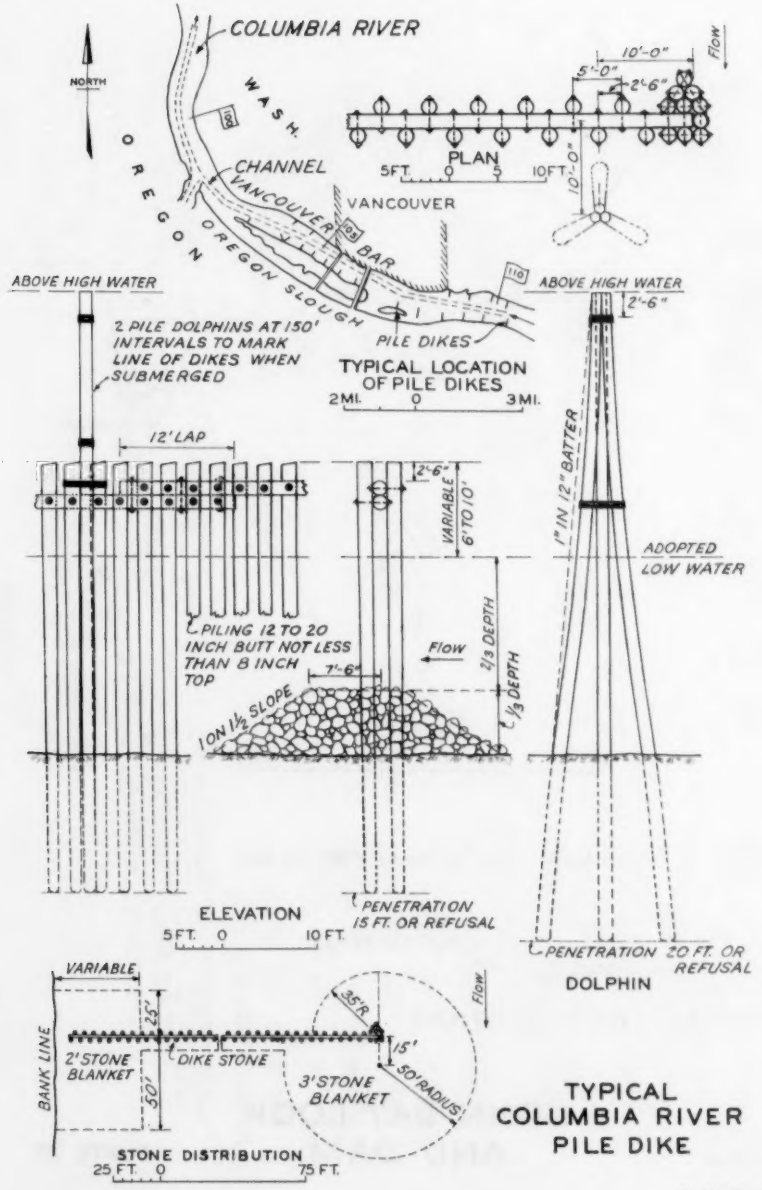




TYPICAL LOCK SECTION

# JOHN DAY LOCK AND DAM

F.G.B.-1958

PLATE VI




# TYPICAL COLUMBIA RIVER PILE DIKE

PLATE VII

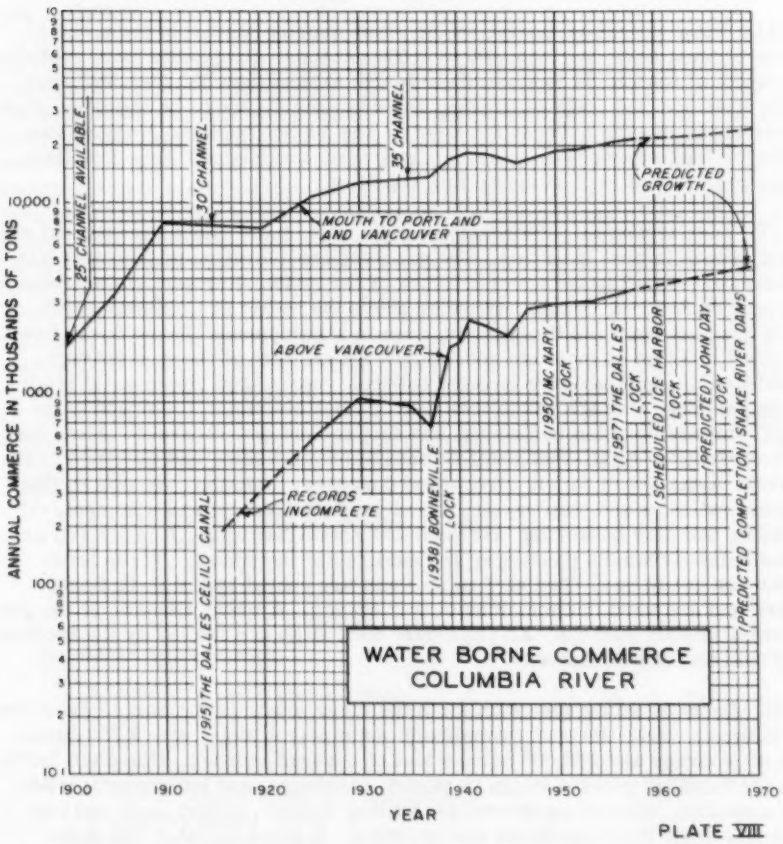

ANNUAL COMMERCE IN THOUSANDS OF TONS
10,000
1000
100
25' CHANNEL AVAILABLE
30' CHANNEL
35' CHANNEL
PREDICTED GROWTH
MOUTH TO PORTLAND AND VANCOUVER
ABOVE VANCOUVER
RECORDS INCOMPLETE
(1915) THE DALLES CELILO CANAL
(1938) BONNEVILLE LOCK
(1950) MC NARY LOCK
(1957) THE DALLES LOCK
(SCHEDULED) ICE HARBOR LOCK
(PREDICTED) JOHN DAY LOCK
(PREDICTED COMPLETION) SNAKE RIVER DAMS
WATER BORNE COMMERCE
COLUMBIA RIVER
1900
1910
1920
1930
1940
1950
1960
1970
YEAR

PLATE VIII

feet long, with a capacity of 4,600 tons. Approximately one-half of these barges are dry-liquid vessels, designed to carry liquid cargo in the hull and/or dry cargo, usually grain, in a deckhouse. Tugs vary from 27 to 113 feet in length and from 165 to 4,000 horsepower. A new river barge 75 feet wide, 295 feet long, with a loaded draft of 15 feet was recently launched.

11. Tows currently moving on the river usually consist of from one to three barges, carrying from 500 to 5,000 net tons. Bonneville Lock restricts the width of tows to 74 feet and their length to less than 500 feet, including tugs, if delays and costs of breakup and reassembly are to be avoided. Fast water in the stretch between the head of The Dalles Pool and McNary Dam calls for high horsepower to provide upstream propulsion. John Day Dam, when constructed, will alleviate this condition.

12. Port facilities. - Increasing traffic flow on the river has spurned related growth in port facilities. The greatest concentration of facilities has, of course, developed in the lower reaches of the river. Farthest downstream is the port of Astoria, Oregon, 14 miles above the mouth of Columbia River. Here the port operates three piers, open to all shippers. These docks have nearly 6,200 feet of moorage and 715,000 square feet of covered storage. Facilities are available for general exports, coastal and intercoastal trade, grain handling, storage and processing, lumber shipment, oil-products receipt, storage and ship bunkering, and fish receipt and shipment. Also, the port operates a small-boat basin to accommodate fishing and pleasure craft. Private docks serve widely diversified purposes, including fish processing, marine railways and ship repair, forest products receipt and shipment, oil handling and ship bunkering, and sand and gravel handling. A ferry slip provides trans-Columbia passenger and automobile movement. Three local docks are not in use. Two at Tongue Point are owned by the U. S. Coast Guard and are used in operations of that agency. A wheat elevator at the port stores 1 million bushels. All important dock sites are served by the Spokane, Portland and Seattle Railway.

13. Some 66 miles upstream from the river mouth is the port of Longview, Washington. Here the port commission maintains a wharf with 2,250 lineal feet of moorage and 235,000 square feet of covered storage. Complete facilities for handling general cargo in export, coastwise, and intercoastal trade are available. Special equipment for loading lumber, export logs, and timbers and other forest products are provided. A grain elevator can store 5,365,000 bushels. Equipment is available for handling petroleum products. Two of the world's largest, integrated, wood-using industries are in Longview. Each mill owns two wharves equipped for the shipment of lumber and other wood products and for receipt of raw materials and supplies. All five docks are served by a belt-line railroad which connects with four transcontinental railways.

14. Vancouver, Washington, is on the north bank of Columbia River 4 or 5 miles upstream from its confluence with the Willamette and 106 miles above the mouth of the Columbia. The port operates two general public docks with total moorage space of 2,390 lineal feet. Covered storage and transit sheds total over 110,000 square feet. Cargo-handling facilities for export and import trade are furnished. Oil is handled at one terminal. The grain dock and

elevator are operated privately. Capacity of the grain elevator is 3,000,000 bushels. There are 10 other docks in the harbor area. Of these, five specialize in handling forest products; one receives and ships sand and gravel; one is a U. S. Coast Guard station; two are shipyard docks held inactive by the U. S. Maritime Commission, and one provides moorage and repair facilities for a major upriver towing company. The port is served by four transcontinental railroads.

15. Portland, largest city in Oregon and one of the major ports of the Pacific Coast, is about 110 miles from the Pacific Ocean, on both banks of Willamette River about 9 miles above its junction with the Columbia. Responsibility for port of Portland activities is divided between two public agencies. The port commission, a state agency, operates two drydocks and associated repair facilities and docks on Swan Island, does dredging required to keep the docks in the harbor accessible to deep-sea vessels, and administers operation of Portland International Airport and other public aviation facilities. The Commission of Public Docks operates three city-owned terminals, consisting of 9 piers which, together, offer more than 2 miles of mooring space. These public terminals have 1,250,000 square feet of covered warehouses and 1,200,000 square feet of open storage. Modern loading and unloading facilities are offered for all types of cargo. New equipment transfers grain from barge to ship or storage at the rate of 12,000 bushels per hour. Grain elevators have a combined capacity of 7,500,000 bushels. Other dock and pier installations are shown in the following tabulation:

Publicly owned docks:

- 2 shipyard outfitting docks (not in use)
- 3 fire-boat slips
- 1 dredge and barge mooring, Corps of Engineers
- 1 harbor patrol landing
- 1 U. S. Coast Guard station

Privately owned docks:

- 6 berth line and general cargo
- 5 grain receipt and cereal shipping
- 11 petroleum products receipt and shipping
- 8 sand and gravel operations
- 8 marine engineering and ship repair
- 2 ship scrapping docks
- 6 tug and barge moorages
- 9 lumber products shipment[1]
- 4 food products[2]
- 3 miscellaneous (cement, creosote, salt)

[1]Of the lumber docks, 7 are inactive due to the shift of lumber production to points in the upper Willamette Valley and Oregon coastal areas.

[2]Cargo no longer moves over these docks, which are used for food warehousing.

16. All major docks and wharves in the harbor are reached by four major transcontinental railroads as well as the Southern Pacific Railway which serves Oregon points and California coastal cities. Photograph No. 2

shows most of the port of Portland.

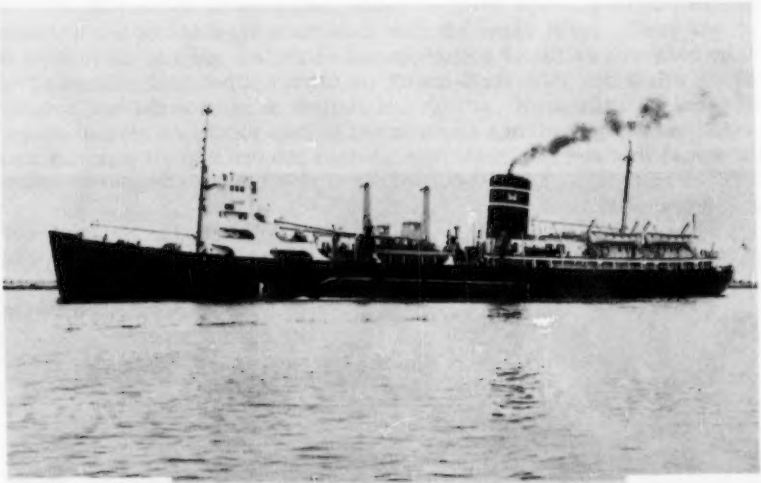
17. Along the Columbia below Vancouver and Portland are several small ports whose facilities are intended to receive materials or ship products of particular industries. Four such lower-river points use their docks entirely for receiving fish. No appreciable amounts of finished products move by water. At six other locations, docks and moorages were built to receive logs and ship lumber. Due to far-advanced harvesting of old-growth timber close to the river, three of these lumber ports are inactive, and it is doubtful that future availability of second-growth timber will result in renewal of shipments of rough lumber. Much more likely is the use of such raw materials in the production of more highly-finished materials within the area. Other lower-river activity results from the assemblage of log rafts at numerous log dumps along the main stem and its tributaries.

18. Above Vancouver as far up Columbia River as its juncture with the Snake, cargo-handling facilities have been built. The port of Camas-Washougal several years ago built a dock and warehouse which have been heavily damaged by spring freshets. No cargo now moves over this dock. Aside from log dumps and raft moorages, no facilities exist between Washougal and Bonneville Dam.

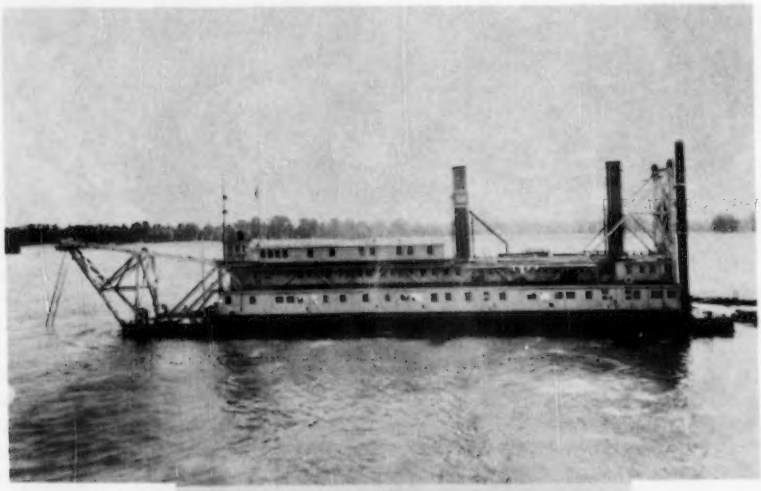
19. In Bonneville Pool, many log dumps and log-handling areas are found. Above The Dalles, timber from the eastern slopes of the Cascade Mountains is processed locally rather than rafted down the Columbia. At The Dalles, a wharf and warehouse installation capable of accommodating oceanborne traffic was completed about 20 years ago. However, because the channel above Vancouver has been maintained only at barge navigation depths, this facility has remained virtually unused. About one-half of the storage area has deteriorated to a point where it is not usable. Two petroleum product depots receive gasoline, oil, and related products for wholesale distributors in the area.

20. The newly-formed The Dalles Pool, beginning at the dam 3 miles upstream from The Dalles and extending about 23 river miles up the mouth of John Day River, does not at present have any cargo facilities. Two wheat elevators, one each at Rufus and Biggs, receive by truck and dispatch by rail to Portland elevators.

21. The stretch of river from the head of The Dalles Pool to the foot of McNary Dam, some 76 river miles long, provides open-river navigation involving several rapids which require the use of powerful tugs to propel tows. The authorized John Day Dam will, when built, provide slack water throughout this river reach. Three ports have developed on this stretch. At Arlington, Oregon, is a grain elevator with a capacity of 300,000 bushels. Near Paterson, Washington, are two wheat elevators storing 1,210,000 bushels. Umatilla, Oregon, the port for Pendleton, Oregon, has one grain elevator of 335,000-bushel capacity, three petroleum wharf and storage plants with a total capacity of nearly 9,250,000 gallons, and an aqua-ammonia plant with storage for 450,000 gallons.



DREDGE RIDDLE. - Hopper Type; Capacity, 3060 cubic yards; Built in 1947; Steam-Electric Powered; Propulsion H.P. 6000; Pump H.P. 2-1150; Suction, 30 inches; Discharge, 28 inches; Maximum Dredging Depth, 62 feet; Minimum Dredging Depth, 22 feet; Estimated capacity per hour, 1032 cubic yards; Steel Hull; Length overall, 351' 6-1/8"; Beam, 60'0"; Depth, 30'0"; Draft maximum, 24'3-1/8"; Minimum, 19'0".



DREDGE HULTNOMAH. - Hydraulic Type; Built in 1913; Steam powered; non-propelled; Pump H.P. 1000; Suction and Discharge diameter, 24 inches; Maximum Dredging Depth, 52 feet; Minimum Dredging Depth, 14 feet; Estimated Capacity per hour, 535 cubic yards; Steel Hull; Length overall, 266'1"; Beam, [illegible]

PORT OF PORTLAND
Willamette River (Ackroyd Photography, Inc.)

22. The pool above McNary Dam, some 60 river miles in length, extends upstream to and includes the confluence with the Snake River. Here are found most of the storage and water transportation facilities now used on the upper Columbia. Included are ports for Pasco-Kennewick and Walla Walla, Washington, the latter being at Wallula and Attalia. Navigation facilities in this reach include docks and loading installations and three grain elevators with a total capacity of 2,900,000 bushels, petroleum product tank farms with total storage of nearly 58,000,000 gallons, cement storage for about 7,500,000 pounds, nitrogenous fertilizer storage for 2,280,000 tons, aqua-ammonia storage for nearly 900,000 gallons, and asphalt emulsion storage in excess of 1,750,000 gallons.

23. The above paragraphs have outlined the facilities which exist along the Columbia River system and which, for the most part, are now used in handling present-day traffic. Future port developments, of course, will be directly related to growth of commerce on the river system.

## F. Future Development

1. Commerce. - Commerce on the river system may be expected to increase in response to two major stimuli. First, as population[1] and industry grow, the demand for gasoline and other petroleum products, already constituting the greater part of upbound traffic, will increase at least proportionately. Similarly, as industry grows, raw materials — bauxite, alumina, industrial chemicals and ores among others — will be in increasing demand. Heavy industrial products of such new industries will also increase movement downstream by water. Outbound shipments which cannot be expected to participate substantially in this growth of traffic include grain, the product of a relatively fixed amount of dry-farming land, and lumber products, manufactured from a resource which, in the tributary area, is not increasing in volume.

2. The second stimulating factor is the anticipated complete development of the rivers. When all authorized dams and locks are operating, currently estimated to become reality in 1975, slackwater navigation will prevail from Bonneville Dam upstream on the Columbia to Pasco-Kennewick, and on the Snake to 30 miles upstream from Lewiston, Idaho. Large limestone deposits in Snake River canyon, now economically unavailable, can then be made profitably accessible by barge to industry and agriculture. Also, the completed system will allow operators to move larger tows with less power and probably at higher speeds. This change may be expected to lower operating costs sufficiently to allow lower tonnage rates. Consequently, larger proportions of existing tonnages may be attracted to water transportation. For these reasons, a Corps of Engineers' navigation benefit study estimates that tonnage carried on the Columbia system above Vancouver would increase

1. The 1950 populations of those states and portions of states considered tributary to the Columbia River system totaled 3,247,000 according to the U.S. Bureau of the Census. Estimates made for the same area by Bonneville Power Administration, using methods considered careful and conservative, place the resident population in the year 2000 at approximately 7 million persons.

from approximately 1,460,000 tons in 1955 to nearly 9,860,000 tons in 2000 A.D.

3. Currently, a large but undetermined part of traffic traveling Columbia stretches above Bonneville Dam arrives in or leaves the Pacific Northwest via the Columbia River. Thus, of approximately 11 million tons of seaborne traffic entering or leaving the Columbia in 1955, something in excess of 1 million tons originated or terminated above Bonneville Dam. Such traffic may be expected to grow at a more rapid rate over the coming years than that entering or leaving the much more industrially mature Portland-Vancouver and lower Columbia districts. That being the case, a reasonable rough estimate of total seaborne tonnage entering or leaving the mouth of the river by the year 2000 would appear to fall in the range between 25 and 30 million tons annually.

4. Increased tonnage in the lower river will probably be accommodated by increased facilities at the already-existing major ports. On the other hand, traffic moving above Bonneville by the year 2000 could not be accommodated by the ports now existing.

5. The Corps of Engineers in their navigation benefit study estimate that by that time numerous new upriver ports will have been established. These ports are tentatively located in the following tabulation:

| River | Pool | Port | River mile above mouth |
|---|---|---|---|
| Columbia | Bonneville | Cascade Locks, Ore. | 151 |
| do | do | Wind River, Wash. | 157 |
| do | do | Hood River, Ore. | 169 |
| do | do | Bingen, Wash. | 169 |
| do | do | Klickitat, Wash. | 180 |
| do | The Dalles | Biggs, Ore. | 209 |
| do | do | Maryhill, Wash. | 210 |
| do | John Day | Roosevelt, Wash. | 244 |
| do | do | Boardman, Ore. | 270 |
| do | McNary | Mouth of Snake River, Wash. | 352 |
| Snake | Ice Harbor | Page, Wash. | 19 |
| do | Lower Monumental | Lyons Ferry, Wash. | 59 |
| do | Little Goose | Central Ferry, Wash. | 83 |
| do | do | Almota, Wash. | 104 |
| do | do | Wawawai, Wash. | 111 |
| do | Lower Granite | Lewiston, Idaho | 140 |
| do | do | Clarkston, Wash. | 140 |
| do | Asotin | Lime Point, Idaho | 170 |

6. Most of these tentative ports on the rivers, particularly above Boone-ville Pool, will occupy sites where virtually no population exists today. Their establishment at strategic port sites and in proximity to power generation will mark the economic coming of age of a river resource which heretofore has served human needs only in a relatively minor way.

7. Maintenance procedure. - Recognizing the rapidly-growing problems of maintaining the channel to authorized dimensions and the possibility of increasing dimensions for future traffic, the Corps of Engineers is studying possible improvements in maintenance procedures. Deposit of the bulk of the spoil along the river banks is no longer adequate, and better methods must be adopted.

8. On eastern rivers such practices have been revised to reduce costs and retain project depths for longer periods. A system of transfer points has been used where excavated spoil is deposited and then transferred by a relatively-fixed unit to onshore disposal sites. The bed material in Columbia River is fine sand with an average median grain size of 0.38 mm. Such material settles rapidly and is, therefore, adaptable for sump-rehandling methods. The length of transfer basins to contain the materials would depend upon river velocities. Such procedure is believed to be particularly adaptable to Columbia River and can be accomplished by using (a) hopper dredges exclusively for main channel dredging, and (b) pipeline dredges as transfer units at selected locations along the channel. Some advantages of this procedure are: (1) As river traffic increases, it becomes more difficult and hazardous to operate pipeline dredges with floating pipelines in the channel, but hopper dredges can operate and avoid traffic easily and can remove the more critical shoals first without traffic hazards; (2) pipeline dredges can operate more efficiently in a sump due to the larger volume of material available with a minimum of pipeline moves; (3) dredges can be used longer each year, reducing plant costs and providing full project depths for a longer period; (4) dredged material will be completely removed from the river and will not contribute to later downstream shoaling; and (5) dredged material will be used for land enhancement.

9. Benefits will materialize as reductions in unit costs through more effective use of equipment, increased land values, and reductions in dredging requirements in subsequent years. These benefits should eventually offset increased costs due to the double dredging operation. Also, the first cost of providing transfer basins, when amortized over a long period will not be a material factor in over-all cost comparisons.

10. Actual locations of transfer points will require engineering and economic analysis. Procedures will need to be established for disposal on shorelands. Sometimes the Government has purchased flood-plain lands and constructed a system of dikes to retain dredged material. It would be advantageous to improve lands benefiting port or diking districts under partnership, cost-sharing arrangements.

11. Navigation channels. - Local interests have asked Congress to authorize a review of the present channel from Vancouver, Washington, and Portland, Oregon, to the sea to determine the feasibility of increasing the depth from 35 to 40 feet and the width from 500 to 750 feet to accommodate larger vessels and avoid restriction of future development of the area. The portion of the river between Vancouver and Bonneville Dam, although authorized and excavated to 27 feet deep and 300 feet wide, was never used by deep-draft vessels. Accordingly, it was allowed to shoal to the presently maintained 15-foot depth, which is sufficient to accommodate present barge traffic. Local

interests recently requested that the channel be restored to 27 feet in depth and stated that sufficient potential commerce is in prospect to justify economically the return to project dimensions. Dredging has started and, under present plans, will be complete in 3 years. In Booneville Pool several rock obstacles require removal to provide a more usable deep-draft channel. This work is also scheduled for completion in 3 years.

12. A serious difficulty for deep-draft navigation exists at Bonneville Dam. The irregular river channels, both up and downstream from the locks, cause undesirable currents that will necessitate tug assistance to maneuver deep-draft vessels safely. This problem is being studied, including the feasibility of constructing a deep-draft lock corresponding in other dimensions to the upstream locks at The Dalles and McNary projects.

13. Locks and dams. - Improvements required to provide slackwater navigation include John Day Dam on the main stem of the river, Lower Monumental, Little Goose, and Lower Granite on the lower Snake River; and Priest Rapids, Wanapum, and Ben Franklin Dams on the Columbia above Pasco.

14. Construction of John Day will start in 1958 and the three remaining Snake River dams have been authorized by Congress. Priest Rapids Dam is under construction, and Wanapum and Ben Franklin are under consideration. Provision for future locks will be required at these two sites.

15. With this combination of structures, a channel of adequate width and 14 feet deep will ultimately be available from the head of McNary Pool to Rock Island near Wenatchee on the Columbia, as well as 15 feet from The Dalles to Lewiston. Above Lewiston, current studies of multipurpose projects include slackwater navigation in the stretch between Lewiston and Lime Point, Idaho.

## SUMMARY

The following table demonstrates the stature of the Columbia River in comparison with other west coast ports as to waterborne commerce:

Total waterborne commerce for 1956
(short tons)

| Port | Foreign | Foreign, coastal, and internal (total) |
|---|---|---|
| San Francisco, Calif. | 8,080,000 | 36,927,000 |
| Columbia River, mouth to International Boundary | 4,750,000 | 21,574,000 |
| Los Angeles, Calif. | 5,600,000 | 19,601,000 |
| Portland, Ore. | 2,400,000 | 13,789,000 |
| Seattle, Wash. | 1,800,000 | 13,652,000 |
| Long Beach, Calif. | 3,400,000 | 7,035,000 |

Since earliest river entry, 166 years have elapsed. Until 1850, however, navigation contributed little to the regional economy. During the past century the Columbia, formerly untamed and barely navigable, has become one of the world's important water channels and harbors. A year-round channel 35 feet deep is available from the ocean to Portland and 30 feet to Vancouver. Above Vancouver, a channel of sufficient dimensions for modern barge traffic exists all the way to Pasco and Kennewick.

As the remaining fertile, relatively untapped area surrounding the Columbia matures, the river's navigation facilities and potential will contribute ever-increasingly to the economic growth of the Northwest.

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House Document No. 531, 79th Congress, 2d session, October 1, 1948.

## PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Divtsion during 1957.

### VOLUME 83 (1957)

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(SA4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1471(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY8), 1493(BD2).

### VOLUME 84 (1958)

JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1)[c], 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(HY2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST2), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW2), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST3)[c], 1788(IR3), 1789(WW4).

[c] Discussion of several papers, grouped by divisions.

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